Proceedings of the
Eighth International Conference on
Fibre-Reinforced Polymer (FRP) Composites
in Civil Engineering

14 - 16 December 2016
Hong Kong, China

Official Conference of the
International Institute for FRP in Construction (IIFC)

CICE 2016

Editors: J.G. Teng and J.G. Dai
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Edited by
J.G. Teng and J.G. Dai
The Hong Kong Polytechnic University

Organized by
Department of Civil and Environmental Engineering
&
Research Institute for Sustainable Urban Development (RISUD)

The Hong Kong Polytechnic University
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Mini-symposium on Near-Surface Mounted FRP for Structural Strengthening
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Sena-Cruz, Jose University of Minho Portugal

Mini-symposium on Seismic Retrofit of RC Structures with FRP Composites
Ilki, Alper Istanbul Technical University Turkey
Ferrier, Emmanuel University of Claude Bernard Lyon 1 France

Mini-symposium on Prestressed FRP for Strengthening and New Constructions
El-Hacha, Raafat University of Calgary Canada
Rojib, Hothifa University of Calgary Canada

Mini-symposium on Strengthening of Steel Structures with FRP
Deng, Jun Guangdong University of Technology China
Yang, Yongxin Central Research Institute of Building and Construction Co., Ltd, MCC Group China

Mini-symposium on FRP-Concrete Hybrid Structures
Feng, Peng Tsinghua University China
Yu, Tao University of Wollongong Australia

Mini-symposium on FRP Sandwich Structures in Bridge and Building Construction
Keller, Thomas Swiss Federal Institute of Technology, Lausanne (EPFL) Switzerland

Mini-symposium on Structures Incorporating FRP Composites under Impact/Blast Loading
Remennikov, Alex University of Wollongong Australia
Huo, Jingsi Huaqiao University China

Special Session on FRP Composites in Spatial Structures
Jiang, Tao Zhejiang University China
Gattas, Joe University of Queensland Australia

Special Session on Natural Fibre Composites in Construction
Xian, Gui-Jun Harbin Institute of Technology China
Yan, Libo Technical University of Braunschweig Germany
The Eighth International Conference on Fibre-Reinforced Polymer (FRP) Composites in Civil Engineering (CICE 2016) was held at The Hong Kong Polytechnic University (PolyU), Hong Kong, China on 14-16 December 2016. It marked the 15th anniversary of the CICE conference series, which is the official conference series of the International Institute for FRP in Construction (IIFC). Since its launch in 2001 in Hong Kong, the CICE conference series has previously travelled to Adelaide (2004), Miami (2006), Zurich (2008), Beijing (2010), Rome (2012) and Vancouver (2014). The CICE 2016 conference was jointly organized by the Department of Civil and Environmental Engineering (CEE) and the Research Institute for Sustainable Urban Development (RISUD) of PolyU.

The first CICE conference, held in 2001, was organised by the same department, which was then known as the Department of Civil and Structural Engineering, and also had Prof. J.G. Teng as the Chair of the Organizing Committee. It was at CICE 2001 when a meeting was called by Prof. Teng to explore the establishment of a world-wide learned society for the FRP-construction community, which led to the founding of the International Institute for FRP in Construction (IIFC) in 2003. The CICE conference has been associated with the IIFC as its official conference series since 2003, and the IIFC is now very firmly the premier international learned society for the FRP-construction community. Given this background, it was a special pleasure to have CICE 2016 held on the PolyU campus to commemorate its 15th anniversary when PolyU celebrates its 80th anniversary.

Following the well-established tradition of the series, CICE 2016 provided an international forum for all concerned with the application of FRP composites in civil engineering to exchange recent advances in both research and practice and strengthen international collaboration for the future development of the field. The conference programme comprised about 240 presentations, including 8 Keynote Lectures, 7 Mini-symposia, 2 Special Sessions, and the IIFC Best PhD Thesis Competition in two sessions, in addition to presentations arising from free submissions. The IIFC Best PhD Thesis Competition, which is a new scheme launched recently by the IIFC as a component of the CICE conference, aims to promote research excellence among our new generation of researchers. The proceedings of the conference consist of two parts: (i) a printed volume that contains the abstracts of all papers accepted by the conference; (ii) a USB disk that contains all full papers (or abstracts if full papers were not submitted). It is hoped that this hybrid format can strike a good balance between ease of access and size of the volume. The papers were contributed by researchers from 31 countries around the world and cover a wide range of themes as indicated by the mini-symposium/special session titles and section headings (see Table of Contents).

The success of the conference was due to the support and efforts of many individuals and organizations. On behalf of the Organizing Committee, we would like to thank all authors for their careful preparation of the manuscripts, all Keynote and Invited Speakers for sharing their work and insight at the conference, and all mini-symposium/special session organizers for mobilizing researchers in their specialised areas to attend the conference. All papers submitted for the conference were reviewed by members of the International Scientific Committee and the Organising Committee as well as other experts in the field. The two Committees, as well as the mini-symposium/special session organisers, also generated widespread interest in and support for the conference. We are grateful to all of them for their important contributions to the conference.
In addition to sharing the paper review task, members of the Organising Committee were also most generous with their time in the organization work. As Chairman and Co-Chairman of the Organising Committee, we are indebted to all of them. Special thanks go to the Executive Members of the Organising Committee, including Dr Chiara CECCATO, Dr Wan-Yang GAO, Dr Guan LIN, Mr Yi-Nan YANG and Miss Anisha Tsang (Conference Secretary), who all toiled through a protracted period to ensure the success of the conference.

We have had the pleasure of working closely with the IIFC, the official sponsor of the CICE conference series. The selection of papers for the Best Paper Awards and the IIFC Best PhD Thesis Award Competition were organised mainly through the efforts of the IIFC. We are grateful to the IIFC, particularly its Executive Committee under the leadership of Prof. Jian-Fei Chen, the IIFC President, for their strong support and valuable advice during the organization of the conference. It has been our pleasure to contribute to the continued success of the IIFC.

Thanks are also due to Prof. William Lam, Head of the Department of Civil and Environmental Engineering, for his support to the conference, including the provision of technical and secretarial support at the conference. We are also grateful to the wider community of PolyU for all the support we have received from them.

This conference was held at a time when PolyU had already launched a year-long celebration of its 80th anniversary. The university was established as a Government Trade School in 1937, and has evolved over the past eight decades into one of the top universities in Asia with internationally recognised excellence in civil engineering and many other disciplines. We were pleased to dedicate this conference to the 80th anniversary of PolyU.

Jin-Guang TENG
Chairman, Organizing Committee

and

Jian-Guo DAI
Co-chairman, Organizing Committee
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2. **Durability of FRP Rebars in Aggressive Environments**  
   Brahim Benmokrane and Ahmed H. Ali  
3. **Longevity Design of Major Structures Strengthened/Reinforced with FRP Composites**  
   Zhishen Wu (Recipient of the IIFC Distinguished Lecture) and Xin Wang  
4. **FRP Sandwich Structures in Bridge and Building Construction**  
   T. Keller  
5. **Bio-Based FRP Composites – Do They Have a Place in Construction?**  
   A. Fam and K. Mak  
6. **Future Building – Carbon for Bridges, Roofs and Facades**  
   M. Schlaich and Y. Liu  
7. **Reducing the Cost of FRP RC Research and Development**  
   D.J. Oehlers and P. Visintin  
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Keynote Lectures
DEBONDING FAILURES IN FRP-STRENGTHENED RC STRUCTURES: RECENT ADVANCES AND RESEARCH NEEDS

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ABSTRACT

The external bonding of fibre-reinforced polymer (FRP) reinforcement to reinforced concrete (RC) structures is now widely accepted as a mainstream strengthening technology. In such FRP-strengthened RC structures, the bonded interface between FRP and concrete plays a key role and debonding of the bonded FRP system from the concrete substrate due to interfacial shear and/or peeling stresses often governs their strength. Although a great number of studies have been undertaken on these structures, reliable numerical simulation of debonding failures has only become possible in recent years. This presentation will provide a review of recent advances in understanding and predicting debonding failures in FRP-strengthened RC structures; it will also examine the key research needs in the area. Special attention will be given to the major challenges encountered in the numerical simulation of debonding failures in FRP-strengthened RC structures and the appropriate use of dynamic solution methods in capturing the complex debonding failure process.

KEYWORDS

FRP, RC beams, strengthening, debonding, cracking, finite element method, span-back, dynamic method, crack band model.
DURABILITY OF FRP REBARS IN AGGRESSIVE ENVIRONMENTS

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ABSTRACT
Fibre-reinforced-polymer (FRP) bars are being used increasingly as internal reinforcement in concrete structures. Despite research and product certification, FRP durability—especially under severe environmental conditions—stands out as the most critical research topic. The lack of appropriate data poses a major obstacle to broader acceptance and mass implementation of FRPs. This paper summarizes the most significant research work published on durability related to FRP internal reinforcement in concrete members. The durability of this type of reinforcement has been extensively investigated over the last two decades. A comprehensive review of the literature, including the degradation mechanisms, accelerated tests for long-term performance, and the effects of environment parameters such as moisture, salt solutions, and alkaline on FRP durability, will be presented and discussed.

KEYWORDS
Durability, FRP bar, environment effect, long-term behavior, reinforced concrete.

INTRODUCTION
The corrosion of steel in concrete has been identified as one of the leading causes of deterioration and structural deficiency (Figure 1a). Repair and restoration costs in the US, Canada, and the majority of European countries constitute a high percentage of total national expenditures on infrastructure. Estimates indicate that the United States spends billions of dollars annually to repair and replace bridge substructures such as piers ($2 billion) and marine piling systems ($1 billion) (Benmokrane et al. 2016). In addition, the U.S. Federal Highway Administration (FHWA) estimates that eliminating the nation’s bridge deficient backlog by 2028 would require an investment of $20.5 billion annually because of corroded steel reinforcement. Canada’s deficit for its municipality infrastructure, which represents 70% of the country’s total infrastructure, was estimated to be $60 billion in 2004, and is expected to grow at $2 billion per year. A relatively recent solution, receiving global attention, is the use of fiber-reinforced-polymer (FRP) composites as cost-effective alternatives to conventional construction materials. Due to their corrosion resistance, light weight, and high strength, FRPs have been widely used in civil infrastructure throughout the world for the last 20 years (Ali et al. 2016). FRP composite materials, consisting of strong, stiff fibers (such as carbon, glass and aramid) impregnated with polymeric resins (such as polyesters, vinyl esters, or epoxies) are being increasingly used in civil-infrastructure applications. Even if FRP reinforcement seems to be a promising solution for corrosion problems, the main issues confronting designers and engineers with respect to using FRP composite materials are their durability qualities and long-term behavior, particularly in civil infrastructure subject to severe environmental conditions. Figures 1(b) and 2 show typical GFRP reinforcing bar and its use in a concrete deck slab, respectively. Despite the fact that the structural integrity of composite bars has been certified, the industry still needs long-term durability data for the material in order for FRP to gain wider acceptance for use in infrastructure applications (Harries et al. 2003). Compared to the large amount of work on the durability of FRP composites originating in the aerospace, chemical, and military industries, there is only a small but growing amount of research concerned with the durability of FRP composites in civil-infrastructure applications. This paper summarizes recent research on various aspects of the durability of FRP composites for use as internal reinforcement for concrete structures. The main topics investigated are related to the effect of different environmental conditions on FRP reinforcement, with the paramount issues being the effects of moisture, alkalis, and saline environments. Most of these parameters have been extensively investigated, and the results can be found elsewhere (Benmokrane et al. 2006, 2011, 2016; Ali et al. 2015; Almusallam et al. 2013).
OVERVIEW

FRPs are heterogeneous combinations (fibre and resin matrix) separated by a distinct interface. The constituents maintain their identities, but the combination produces properties and characteristics beyond that of the constituents, yet highly dependent on fibre–resin cohesion. The resin system acts as a matrix bonding the fibres and spreading the load among individual fibres. It also protects them from abrasion, impact, and environmental conditions. The fibre–resin interface is of paramount importance, requiring sound adhesion to ensure optimal load transfer and long-term performance. This multiphase structure should be taken into account when investigating FRP durability.

FRP-bar durability is a concern for a few reasons. First, FRP bars are still “new materials” compared to traditional steel bars. Second, FRP-bar innovation is ongoing with a need to verify the durability of new materials and seek new FRP bars with higher durability. Third, design codes and building/transportation authorities currently specify a 75- to 150-year life: FRP durability needs to be demonstrated.

Unlike steel, FRP bars are resistant to electrochemical corrosion, although the literature indicates that they could deteriorate due to certain physical (e.g., cyclic or sustained loading, moisture diffusion, and temperature) and chemical (e.g., alkalinity) effects. The degree or rate of damage/deterioration—and its reduced strength, stiffness, and durability—depends on many factors such as fibre type and volume, resin matrix, fibre–matrix interface, manufacturing process, and exposure environments. Degradation processes in FRP bars are typically denoted as fibre dominated; matrix dominated; and interface dominated.

DURABILITY CONCERNS

Although FRPs are resistant to electrochemical corrosion, which severely affects steel, FRP performance can be degraded due to environmental, physical, or chemical conditions, leading to loss of strength and stiffness. The literature (Robert et al. 2009; Robert and Benmokrane 2010) indicates that the FRP performance deteriorates due to certain physical (e.g., cyclic or sustained loading, moisture diffusion, extreme temperature variations) or chemical (e.g., alkalinity) exposure. The degree of damage/deterioration depends on a variety of factors such as fiber type and volume, resin matrix, manufacturing process (curing rate, thermal micro-cracks, porosity, non-impregnated fibers, void content), and exposure environments (Ali et al. 2015). Furthermore, adding FRP composites to concrete structures complicates the durability performance of FRP-reinforced concrete structures.
due to the combined effect of FRP composites, interface, concrete, and various environmental and mechanical conditions. Hence, the whole process of assessing the durability of FRPs, in association with concrete structures, is a very complex and multidimensional task. The necessity of a broad assessment of the durability of FRP composites in association with concrete, and as an individual material, cannot be overemphasized. The following sections attempt to look at various durability aspects of FRP composites, FRP-reinforced concrete members, and the bond between FRP and concrete. To focus on the complexity of the analysis, our discussion focuses solely on three factors influencing durability: fluids (moisture; chemical solutions), alkalinity/salinity, and freeze/thaw cycles.

**Fluids and Moisture**

Concrete containing internal FRP reinforcement is commonly exposed to alternating wet/dry cycles, natural weathering, and sometimes corrosive media. Even if concrete provides an excellent first line of defense, its permeability is high enough to eventually transport moisture and other corrosive elements to the internal reinforcement. In recent decades, the effect of fluids on the performance of FRP composites has been one of the most studied subjects related to composite durability. In general, the sorption of fluids by FRPs depends on fluid types (water, acids, bases), fluid concentration, temperature, external applied stress, type of fiber and resin, interphase, molding process, and state of the material (damage, curing conditions). Ben Daly et al. (2007) showed that the moisture-diffusion process in pultruded composites and the saturation level attained could be related to the presence of fillers and additives in the polymer matrix. It has been proven that the sorption rate is controlled by the matrix’s chemical structure (degree and type of cross-linking, presence of voids), interface/interphase, and manufacturing process. Consequently, researchers have attempted to control the diffusion process by using resin matrices of lower permeability (Benmokrane 2000), modifying the interphase region by using suitable sizing chemistry, or selecting an appropriate molding process to reduce void content. Moreover, moisture ingress can degrade the resin through chemical attack (hydrolysis) or a drop in glass transition temperature. For this reason, fluids affect dominant matrix properties, such as the transverse and shear strengths of FRP composites, and these properties decrease more and more with increased exposure time and temperature.

Glass fibers are particularly sensitive to fluid ingress, since they react to chemical and physical attacks. The level of degradation depends on fiber composition, fluid type and concentration, and exposed temperature. Extensive studies have been conducted on this topic (Chen et al. 2007; Robert et al. 2011, 2010, Robert and Benmokrane, 2010, 2013, 2015). Carbon fibers are not affected by fluid ingress, but the resin matrix is usually affected, the performance of this composite is nevertheless affected. In the case of unidirectional carbon composites, this usually leads to reduced compressive and shear strength, with little impact on tensile strength, since it is especially dominated by the fibers, which are not affected by fluids. Hancox and Mayer (1994) reported minimal weight gain and strength loss for carbon/epoxy specimens exposed to 65% humidity for over four months and to boiling water for over three weeks. Aramid fibers are affected by fluids, mostly at higher temperatures. AFRP composites saturated in water have been reported to lose 35% of their flexural strength at room temperature and up to 55% if stressed and under wet/dry thermal cycles.

**Alkaline and Saline Environments**

The concrete environment is characterized by high alkalinity, with a pH between 12 and 13 depending on the concrete design mixture and the type of cement used (Benmokrane et al., 2006). This alkaline environment damages glass fibers through loss in toughness and strength and increased embrittlement. In general, carbon fibers exhibit the best alkaline resistance, followed by aramid and glass fibers. Glass fibers are damaged due to the combination of two processes: (1) chemical attack on the glass fibers by the alkaline cement environment, and (2) concentration and growth of hydration products between individual filaments. Fiber embrittlement results from nucleation of calcium hydroxide on the fiber surface. The hydroxylation can cause fiber surface pitting and roughness, flaws that severely reduce fibers properties in the presence of moisture. In addition, the calcium, sodium and potassium ions in the concrete pore solution are highly aggressive towards glass fibers. Therefore, the degradation of glass fibers is not only caused by the high pH level, but by the combination of alkali salts, pH, and moisture. Aramid fibers show strength degradation in an alkaline environment. Kevlar 29 exposed to 10% sodium hydroxide solution for 1000 hours loses 74% of its strength. High-modulus aramids such as Kevlar 49 demonstrates better alkaline resistance. Carbon fibers are not supposed to be affected by alkaline solution at any concentration or by water temperatures up to boiling (Judd 1971; Benmokrane et al. 2016). Judd (1971) reported that carbon fibers were resistant to alkaline solutions at all concentrations and all temperatures up to boiling. Carbon tow immersed for 257 days in a very basic 50% sodium hydroxide solution showed variations in strength and elastic modulus of only around 15%. Katsuki and Uomoto (1995) used electron probe microscopy to track the ingress of alkali ions (sodium ions) into aramid-, carbon-, and glass-reinforced vinyl-ester rods. Sodium ions penetrated into the GFRP radially over time. No degradation was noticed in AFRP or CFRP rods immersed for 60 days compared
to GFRP rods. Chin et al. (2001) used energy dispersive X-ray analysis to observe that appreciable amounts of sodium, potassium, and calcium were found within isopolyester specimens exposed to pore solution at 60°C for 60 days. They also noticed, however, no evidence of ion ingress in the vinyl-ester specimen.

Chu and Karbhari (2002) and Chu et al. (2004) conducted studies on characterizing and modeling the effects of moisture and alkalis on E-glass/vinyl-ester composite strips at different temperatures (23°C, 40°C, 60°C, and 80°C). The tensile-strength degradation levels between 35% and 62% of the initial strength. The test results of Gaona’s durability study (2003) conducted on bare GFRP bars showed that the tensile strength of the tested GFRP bars decreased over time when the bars were in direct contact with solutions simulating concrete interstitial solution. Losses up to 24% were measured for bars conditioned in an alkaline solution with a high pH value (12) at 35°C for 50 weeks. Wang (2005) conducted 330 accelerated aging tests on different diameters of E-glass/vinyl-ester reinforcing bars. The samples were conditioned in alkaline solution (pH of 12.6–12.8) and distilled water at 23°C, 40°C and 60°C for 150 and 300 days. No fiber defects were observed in any of the 150-day samples, although matrix cracks were noted in the 300-day samples. Al-Zahrani (2007) investigated degradation in the residual tensile strength of three types of GFRP bars in aggressive solutions. Bare bars were conditioned in four solutions (alkaline, alkaline + seawater, alkaline + sabkha, and acidic) at three different temperatures for 3–12 months. The maximum reduction in the tensile strength ranged between 27% and 71% in the alkaline environment and sabkha at 60°C. In the case of thermal variation and outdoor exposure, the reduction ranged from 5% to 21%. Kim et al. (2008) conducted a short-term durability test on two types of commercially available bare GFRP bars (E-glass/vinyl-ester) under four different environmental conditions (moisture, chloride, alkali, and freeze/thaw cycling) for up to 132 days. In addition to the room temperature (25°C), elevated temperatures of 40°C and 80°C were employed to accelerate the degradation of the GFRP bars. They concluded that an alkaline environment had greater impact on the degradation of the tensile strength of GFRP bars than the other influencing factors.

Robert et al. (2013) studied the mechanical, durability, and microstructural characterization of unstressed glass-fiber-reinforced-polymer (GFRP) reinforcing bars exposed to a concrete environment and saline solutions under accelerating conditions. These conditionings were used to simulate the effect of seawater and deicing salts on GFRP bars. The results revealed no significant differences in the durability of the concrete-wraped GFRP bars, whether immersed in saline solution or tap water; the GFRP bars in saline solution evidenced very high long-term durability. Benmokrane et al. (2016) conducted a recent study on the durability performance of carbon-fiber-composite-cable (CFCC) tendons exposed to elevated temperature and alkaline environment. Specimens were exposed to alkaline solutions for 1000, 3000, 5000, and 7000 hours at elevated temperatures (22°C, 40°C, 50°C, and 60°C) to accelerate the effect of the concrete environment. The durability performance of the CFCC tendons was assessed by conducting tensile tests on the specimens after different exposure times. The test results revealed a 7.17% reduction in tensile strength after 7000 h of immersion in the alkaline solution at 60°C. The tensile-strength reduction was attributed to the development of microcracks in the epoxy resin as the result of existing defects in the material.

**Durability of GFRP Bars Embedded in Concrete**

Few research studies have investigated the durability of GFRP bars embedded in concrete. Al-Zahrani (2007) subjected small concrete prism specimens (10 x 10 x 100 cm) reinforced with a single embedded centrally GFRP bar to continuous wetting at 30°C in potable water, seawater, sabkha solution, or outdoors for 6–24 months. He observed that the strength reductions were much lower than in the case of bare bars. This reduction ranged between 10% and 35% after 24 months of exposure. The researcher concluded that this behavior could be attributed to the limited availability of moisture around the bars and the lower temperature of the conditioning solutions, which was 30°C for the embedded bars and 60°C for the bare bars. Chen et al. (2007) conducted another durability study on GFRP bars in which bare FRP bars and bars embedded in concrete were exposed to five different solutions. The results showed significant strength loss from the accelerated exposure of both bare and embedded GFRP bars, especially with the solutions at 60°C. Continuous immersion resulted in greater degradation than exposure to wetting/drying cycles. In contrast, freeze/thaw cycles combined with solution exposure produced little degradation in the GFRP bars. Robert et al. (2009) conducted a study on the durability of GFRP bars in moist concrete in which sand-coated GFRP bars with a nominal diameter of 12.7 mm were embedded in concrete and exposed to tap water at 23°C, 40°C, and 50°C for 60 to 240 days. The tensile test results showed decreases in tensile strength of 10% and 16%, respectively, compared to the original tensile strength, after 240 days of exposure at 40°C and 50°C.

In a 2005 field study conducted by Mufti et al., concrete cores were taken from five in-service concrete bridge structures across Canada reinforced with GFRP bars and ranging in age from six to eight years. On the basis of a microscopic and chemical analysis involving simulated laboratory studies with alkaline solutions, they concluded that the concerns about the durability of GFRP in alkaline concrete were unfounded. Recently, the performance of the GFRP bars was assessed by conducting tensile tests on the bars extracted from the concrete prisms after...
exposure to different conditions (Almusallam et al. 2013). In addition, scanning electron microscopy was used to investigate the bar degradation mechanism. The test results after 18 months of specimen exposure showed that both the tap water at 50°C and the alkaline solution at 50°C produced the greatest reductions in the tensile strength of the bars. The two field conditions evidenced almost no degradation in the tensile properties of the bars (Almusallam et al. 2013).

Freezing and Freeze/Thaw Cycles

In general, freezing and freeze/thaw exposures do not affect fibers, although such exposure can affect the resin and the fiber/resin interface. Most studies on this subject were conducted on aerospace materials. The literature indicates that freezing and thawing has very limited impact on pultruded FRP composites. Lord and Dutta (1988) produced an extensive review on material degradation due to low temperatures and freeze/thaw cycles. In general, at low temperatures, complex residual stress arises within FRP composites as a result of matrix stiffening and mismatch of thermal expansion coefficients of the matrix and resin as well as of the FRP and concrete. Residual stresses can cause microcracks in the matrix and fiber/matrix interface, which can grow under low-temperature thermal cycling and coalesce to form transverse matrix cracks and fiber/matrix debonding, leading to degradation of FRP composites. The presence of deicing salt under wet conditions with subsequent freeze/thaw cycling can cause microcrack formation and gradual degradation due to crystal formation and increased salt concentration, in addition to moisture effects that include swelling and drying.

In general, the literature shows that unidirectional tensile strengths decreased in the -10°C to -40°C range, whereas the off-axis and transverse strengths may increase due to matrix hardening. Increasing freeze/thaw cycles have been shown to accentuate residual stresses, resulting in increased crack severity and density. An apparent increase in matrix brittleness and decrease in tensile strength has also been reported (Bennmokrane et al. 1998). In contrast, Shao and Koudi (2002) showed that samples from pultruded GFRP sheets saturated in water provided excellent resistance to freeze/thaw cycles. The authors observed nearly no changes in tensile properties of the FRP composites after 564 cycles between 4.4°C and -17.2°C, even when the samples were saturated with water. Mashima and Iwamoto (1993) studied the change in bond characteristics due to freeze/thaw action on nonmetallic reinforcement composed of synthetic fibers such as carbon, aramid, and vinylon. The bond strength was measured by pullout testing after 200 freeze/thaw cycles. Testing of the 10 x 10 x 10 cm cubic specimens began after 14 days of curing in water at 20°C. Four types of CFRP, GFRP, and AFRP rods and steel reinforcing bars were used in the study. It was determined that the bond strength of the glass-, vinylon-, and carbon-FRP rods were not influenced by freeze/thaw cycles, but the aramid-FRP rods (both braided and coiled types) evidenced a gradual reduction in bond strength of up to about 20% with progressive freezing and thawing. Alves et al. (2010) investigated the durability of FRP bonding to concrete elements subjected simultaneously to 250 freeze/thaw cycles and sustained load. The test results revealed that the combined conditions increased the GFRP–concrete bond strength.

DURABILITY OF GFRP-REINFORCED CONCRETE IN FIELD STRUCTURES

In 2004, ISIS Canada, a Canadian Network of Centres of Excellence, launched a major study to obtain field data on the durability of GFRP in concrete exposed to natural environments. The objective of the study was to provide performance data on a GFRP that has been used in several structures across Canada. Concrete cores containing the GFRP were removed from several five- to eight-year-old exposed structures, and the GFRP’s physical and chemical compositions were analyzed at the microscopic level (first round of tests).

Five ISIS Canada field demonstration structures built in five different provinces (Hall’s Harbor Wharf in Nova Scotia, Joffre Bridge in Quebec, Chatham Bridge in Ontario, Crowchild Trail Bridge in Alberta, Waterloo Creek Bridge in British Columbia) exposed to a wide range of environmental conditions were chosen, (see Figure 3). Their selection reflects a wide range of environmental conditions that are representative of the Canadian climate. Three research teams from four Canadian universities independently performed microanalyses of the GFRP and surrounding concrete. They used a variety of analytical methods to (a) investigate whether or not the GFRP in the concrete field structures had been attacked by alkalis and (b) compare the composition of the GFRP removed from in-service structures to that of control specimens, which were saved from the projects and not exposed to the concrete environment (see Figure 4).

Direct comparisons were carried out with “virgin” GFRP rods preserved under controlled laboratory conditions. The results indicate that no deterioration of the GFRP took place in any of the field demonstration structures included in this study, and that no chemical degradation processes occurred within the GFRP due to concrete alkalinity. The overall conclusion was that the GFRP is durable in concrete. The outcome of this work led to the second edition of the CHBDC allowing GFRP as primary reinforcement and prestressing tendons in concrete structures (CSA S6-14).
In 2009, core specimens of GFRP reinforcement (second round of tests) were collected from three structures of the field demonstration projects studied in the earlier rounds of tests in 2004. These three structures are: Hall’s Harbor Wharf in Nova Scotia, Joffre Bridge in Quebec, and Crowchild Trail Bridge in Alberta. Analytical methods including Scanning Electron Microscopy (SEM), Energy Dispersive X-ray (EDX), Optical Microscopy (OM), Differential Scanning Calorimetry (DSC) and Fourier Transform Infrared Spectroscopy (FTIS) were used to assess the condition of GFRP after 10 to 13 years of exposure to concrete. These analyses were performed to provide further information on the durability of GFRP in field environments as opposed to simulated alkaline environments. These conclusions pertain to the durability of GFRP reinforcement that has been used in Hall’s Harbor Wharf, Joffre Bridge and Crowchild Trail Bridge for a period of concrete exposure ranging between 5 and 13 years: 1) Results from OM did not show any sign of gaps or debonding between the GFRP reinforcing bars and the surrounding concrete in all of the three structures examined. The results also suggest that there were no cracks, slits or fissures in the concrete, indicating that the bond was preserved between the materials; 2) The results from DSC indicate that the structure of the polymer matrix of GFRP reinforcement was not significantly disrupted by exposure to the environment. Neither hydrolysis nor significant changes in the glass transition temperature of the matrix took place after exposure to the combined effects of concrete alkaline environment and the external natural environmental exposure for 10-13 years; 3) The FTIS analyses support the results from the DSC analyses and OM examinations; 4) Micrographs from the SEM analyses did not reveal any signs of physical damage to the fibers, polymer or fiber/matrix interface. The result brings confirmation to the fact that the structures were not exposed to the detrimental effects of alkali attack from the surrounding concrete; 5) Accordingly, results from the EDX analyses do not show traces of alkaline solution within the cross-section of the reinforcement for any of the structures considered.

CONCLUSIONS
Applications of FRP rebar in different concrete structures have proven to be very successful to date. Based on investigational findings, the durability performance of FRP materials is very good in comparison with other, more conventional, construction materials. Field studies have proven that Glass FRP (GFRP) bars are durable in concrete environment and in-service structures. GFRP is the preferred material for concrete reinforcement in North America due to its economical competitiveness and durability.
Future Research

The reported studies show that the fibre–resin interface is of paramount importance, requiring sound adhesion to ensure optimal load transfer and long-term performance. Also, there has been very little research on the synergistic effects (e.g., moisture and load) of the long-term properties of FRP bars; synergistic effects are not typically addressed in current codes and testing standards. Significant effort should be undertaken to study the synergistic effects between environmental factors and loads on both the material and structural scales. Analyzing the interaction between moisture/temperature/stress and damage development will help in developing synergistic models incorporating material variability and in assessing the applicability of Arrhenius-type models. The Arrhenius concept assumes that a single mechanism controls material degradation at all temperatures (below the material’s glass-transition temperature). A new degradation model providing for changing degradation mechanisms over time (due to stress or other environmental factors) needs to be developed for better service-life prediction. With new knowledge and data on the durability of FRP bars, the current design factors and stress limits in codes could be reassessed to economically use FRP bars as concrete reinforcement. Finally, recent ongoing efforts into developing and innovating FRP-bar technology has focused on using new fibres (e.g., basalt fibers, ceramic fibers), high-performance resin systems with low permeability, and new materials such as nanocomposites. These advanced materials are expected to yield benefits over existing FRP bars, while being significantly more cost-effective, yet research is needed to establish and assess their durability.

REFERENCES


LONGEVITY DESIGN OF MAJOR STRUCTURES
STRENGTHENED/REINFORCED WITH FRP COMPOSITES

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ABSTRACT

Fiber-reinforced polymer (FRP) composites have been developed as advanced structural materials to achieve high performance and longevity in major structures. However, due to insufficient studies on the evaluation of the service life of FRPs under various conditions, long-term behavior of FRP strengthened/reinforced structures, and the life cycle design of FRPs in major structures has not been well developed. Focusing on the above limitations, the authors’ team has conducted a series of innovative research over the past ten years on developing advanced basalt FRP composites and various stable composite products. Subsequently, are proposing evaluation methods addressing long-term behavior of FRPs under cyclic and sustained load and their enhancement through hybridization, matrix modification, and pretension treatment. Based on the understanding of the long-term behavior of FRP composites, further evaluation of the long-term interface behavior of FRP strengthened concrete structure and their enhancement are herein proposed for the longevity design of strengthening existing RC structures. Meanwhile, to address the longevity design of new construction with FRPs, the philosophy and the methods to realize high performance and longevity of FRP cable supported bridges are introduced.

KEYWORDS

Advancement of FRP, hybridization, durability, damage controllability, lightweight, longevity.

INTRODUCTION

Fiber-reinforced polymer (FRP) composites are widely known for their high strength, light weight, corrosion and fatigue resistant, and ease of construction in comparison with the conventional steel and concrete materials. In the 1980s, the FRP materials began to be used in the field of civil engineering due to the stimulation of “Concrete Crisis”, which was mainly caused by the problems with steel reinforcement corrosion in concrete structures (Wu et al. 2010). In the mid-1990s, since Japan's Kobe earthquake resulted in a great deal of civil transportation infrastructure disasters, FRP quickly developed into an important means for seismic reinforcement. Since then, structural application of FRPs has been widely studied and explored and extensive research has been carried out in the area of critical physical and in-service properties of FRP and their reinforcement applications (Bakis and Bank 2002; Keller 2003; Hollyway 2010; Wu et al. 2014). The incorporation of FRPs has evolved from the initial retrofitting and reconstruction to building new structures. A wide range of FRP products have developed from FRP fabric/plates, to a variety of forms of fiber composite bars/cables, grids and profiles. The range and the forms of application of FRP’s are being more and more widely explored on internal reinforcements, externally bonded reinforcements, and structural shapes and bridge decks. In addition, preliminary studies of FRPs as a separate structural unit, such as FRP tube truss structures, FRP woven structure, FRP cable large span cable-stayed structure, FRP profile Bridge panel and box girders, have also found applications and new grounds. These structures can significantly reduce the weight of the original structure, enhance the durability and long-term safety of the structure, and can reach the span requirements that cannot be achieved by conventional materials (Meier 1987).

However, although in some aspects the structures with FRP reinforcements exhibit apparent advantages in comparison with structures made with conventional materials, the FRP reinforcements still have not become a major structural material and are not yet being used in major engineering constructions. One of the most significant problems is that although FRP is regarded as the superior material for long-term behavior, there is still a lack of understanding the gap between how to properly design and evaluate FRPs in strengthened or reinforced structures based on service life design. Meanwhile, the insufficient study on the high performance to cost of FRP composites,
Aside from conventional CFRPs, and how to control and design their service life according to structural requirements is also a challenge.

On the other hand, from the perspective of the cost minimization of full life cycle (LCC) of infrastructure, understanding how to control and design for proper service life is a critical factor for building a resource-saving, energy saving and environmental-friendly society. The maintenance and upgrading costs in traditional design life (50-100 years) of large infrastructure systems have become a serious obstacle to the sustainable development of nations. The demand of long service cycle of the infrastructure and the problems of steel corrosion continue to result in huge financial losses world-wide. For instance, in mid-1990s, the infrastructure investment budget of Japan reached 50 trillion yen, and the amount of the investment budget has continued to decrease after 2005. In the future this budget can only be maintained at a scale of 30 trillion yen, while it only meets existing project managements, maintenance, repair and update requirements.

Europe, the U.S., Japan and other developed countries have proposed to implement the design philosophy of minimizing the full life cycle cost (LCC), including construction, custody, use, dismantling the economic costs and environmental costs, early stage of project construction or reinforcement of the social costs, etc., in order to reduce the huge costs of infrastructure maintenance management. Long life design (more than 100 years) has been recognized as one of the most effective methods in order to significantly reduce the upgrading costs, while also largely reducing the resource consumption. Therefore, construction based on longevity design can significantly lower the LCC of structures during their real service life, which greatly benefits sustainable development.

By focusing on the above background work on FRP research and applications, and the longevity design demands for major engineering structures, the evaluation and advancement of FRP composites and their strengthened/reinforced structures will be introduced in this paper.

**LONG-TERM BEHAVIOR OF FRP COMPOSITES**

Long-term behavior of FRP composites mainly depends on the fatigue behavior under cyclic load and creep behavior under sustained load. These two key governing factors can control the service life of structures subjected to the cyclic or working load at high stress level. In the following section these two issues are discussed separately.

**Fatigue behavior**

Fatigue behavior is a key issue and sometimes the controlling factor for the service life in several applications of FRPs in construction under cyclic load, such as FRP cables in cable-supported bridges. However, fatigue behavior of FRP differs from conventional metal materials due to the complex composition of fibers, matrix and interfaces. Thus, focusing on the longevity design of structures, for instance, service life from 100 to 300 years, how to properly evaluate the fatigue behavior of FRPs under long cycles and how to consider the environmental impacts on the fatigue life are necessary and form the basis for structural longevity design. The latest research on fatigue behavior of basalt FRPs as pre-stressing members, is herein introduced. The main focus in this research includes the micro fatigue behavior, macro fatigue performance and the consideration of environmental impacts.

**Fatigue degradation mechanism of FRPs**

*Fatigue-SEM testing device*

A system that can simultaneously perform the fatigue loading test and SEM observation as shown in Figure 1 was developed to clearly identify the damage evolution and its mechanism during the fatigue loading. For transferring the fatigue load to the specimens, a pair of steel wedges was attached to the fatigue servo system and pre-tightened with the specimen. Aluminum shims were used between the wedges and the specimens to smoothen the stiffness change between them and to minimize the risk of grip failure during fatigue testing (Zhao et al. 2016). By using this technique the effect of defects and selecting standardized products for fatigue test can also be excluded.

![Figure 1 Fatigue-SEM testing device setup. (a) SEM Servopulser; (b) Layout diagram of gripping device](image-url)
Fatigue damage propagation under different max stress

The damage propagation patterns during fatigue loading vary under different max stress levels. At high stress level, the fracture of the specimens takes place in the form of progressive fiber breaking, similar to static tensile fracture. At modest stress levels, the short matrix cracks propagate transversely over a few fibers, or along the interface of fiber and matrix, resulting in the initiation of progressive interface debonding. Furthermore, at low stress level, only discontinuous interface debonding occurs due to the propagation of micro cracks in matrix (Zhao et al. 2016).

Fatigue damage propagation under different stress ratios

Fatigue failure mode also depends on the stress ratio. Figure 2 shows three typical failure surfaces at the stress level of 75% for specimens of R=0.8, 0.5 and 0.1 respectively. It can be clearly seen that the control mode of damage is a discontinuous interface debonding at the stress level of 75% for R=0.8. However, the failure modes shift into fibers wearing out and even breaking when the stress ratios decrease.

Fatigue behavior under 2 million and 10 million

A bi-linear phenomenological fatigue model (Harik et al. 2002) is adopted to describe the effect of different property degradation rates (Zhao et al. 2016), as shown in Figure 3. It can be seen that the slopes of the linear and bi-linear lines are not consistent, which indicates the three different failure mechanisms under different periods of cycles, as shown in Figure 4. The prediction with data up to 2×10⁶ cycles is lower than that with all data, which indicates that the degradation rate under fatigue load slows down after 2×10⁶ cycles.

Fatigue life prediction by constant life

As a method for predicting fatigue life, CLD diagrams (Figure 5) are linear or nonlinear interpolation schemes between experimentally obtained S–N curves (like Figure 6) in the mean stress (σ_m)-stress amplitude (σ_a) coordinate system. These diagrams are widely used for demonstrating the effect of stress ratio on the fatigue life and generalizing the predictions for arbitrary stress ratios. Lines through the origin represent data for a specific stress ratio (S-N curves). Lines connecting data points corresponding to a specific fatigue life for different stress ratios are also derived to allow (by interpolation between known data) the prediction of S-N curves at additional R-ratios.
Fatigue behavior of macro FRP composites

Based on the aforementioned damage evolution and mechanism of FRPs under cyclic load, fatigue behavior of FRP tendons, which is a typical form in engineering application, were investigated (Wang et al. 2015a). Following the practical use of FRP tendons, firstly, the applied stress ranges varied from 0.05 \( f_u \) to 0.14 \( f_u \), and the minimum stress level remained constant at 0.35 \( f_u \), to simulate the moderate and severe working conditions in service (El Refai 2013). Secondly, the maximum stress levels applied ranged from 0.6 \( f_u \) to 0.8 \( f_u \) to examine the potential maximum stress for BFRP tendons with a constant stress range of 0.05 \( f_u \) (the maximum stress range in cable-supported bridge Wang et al. (2014a)). Slightly different from the above meso mechanism, the failure mode of FRP tendons is mainly caused by the fiber-matrix interface debonding, which is induced by the propagation of micro cracks in matrix. The stress range greatly affects the fatigue life of BFRP tendons even under relatively low stress levels. Specimens, with a stress range limit of 0.05 \( f_u \) (85 MPa), and a maximum fatigue stress level up to 0.6 \( f_u \), survived after two million cycles (Figure 7). The fatigue behavior of BFRP tendon fulfills the structural requirements of cable-supported bridge structures.

Fatigue behavior of FRP under multi field coupling
Under high stress range

Initial interface damage is introduced after corrosion under marine environment (Figure 8). After one week in the salt solution, only discontinuous interface debonding was observed among the regions of fiber breaking within 50000 fatigue cycles. The interface damage became serious with the aging duration. Long and continuous interface debonding occurred in the specimen after six-weeks in the salt solution within 50000 fatigue cycles. Thus, Marine water action destroys the adhesion between the fibers and the matrix. This interface damage could result in early interface debonding during the fatigue loading.
Under practical stress range

Based on the above fatigue research of BFRP tendon, fiber-matrix interface debonding is the main cause of FRP under practical stress range (0.05 f₀). The fatigue strength degradation of BFRP tendon under a marine environment is mainly caused by the debonding of the fiber-matrix due to Si-O-Si chemical bond hydrolyzation (Figure 9). However, experimental results showed that fatigue strength reduction of BFRP tendons aged at relatively low temperature (25 or 40 °C) was limited (less than 10 %).

Under temperature acceleration

SEM analysis (Figure 10) showed that specimens aged at a high temperature (55 °C) for 63 d exhibited larger corrosive areas and more corrosive channels than the specimens in the other groups. The infiltration of hydrones in the salt solution was accelerated by high temperature, leading to more serious hydrolyzation of the Si-O-Si chemical bond between the fiber and the matrix. However, the fatigue strength of specimens aged at 55 °C for 63 d is 0.52 f₀, which still fulfills the requirements of prestressing application.

Figure 9 Mechanism of fatigue degradation  Figure 10 SEM analysis  Figure 11 Fatigue strength prediction

Prediction of fatigue strength

Using Arrhenius equation, the degradation laws of BFRP tendon can be predicted during the entire service life. The values of the fatigue strength of BFRP tendons were predicted to be 0.41, 0.43 and 0.45 f₀ at northern latitudes of 20°, 40° and 60°, respectively, after aging in marine environments for 100 years, as show in Figure 11. These results provide guidance for the fatigue design of BFRP tendons utilized under a marine environment.

Enhancement of fatigue behavior

Due to the high cost, it is difficult to accept CFRP for a wide range of applications in construction, although it possesses superior properties compared with other FRPs. In order to enhance the utilization efficiency of high performance to cost FRP composites, one of the effective ways is to hybridize basalt or glass fibers with high fatigue resistance fibers like carbon. It is apparently shown that the fatigue behavior can be significantly enhanced by hybridization of carbon and basalt fibers. On the contrary, negative effect on the fatigue behavior was observed for hybridization with carbon and glass fibers (Figure 12 and 13). The interfacial behavior between fibers and matrix plays a critical function in fatigue behavior enhancement (Wu et al. 2010).

Figure 12 S-N curves of different FRPs  Figure 13 Mechanism of hybrid effect

In order to enhance the fatigue and anti-corrosion behavior of FRP composites, matrix modification methods, like rubber particle toughening and Nano-kaolinite clay modifying, can be used. The increases of fatigue life of FRP were observed by modified matrix (Manjunatha et al. 2010). As it is clarified above that cracking in the matrix is the initiation of fatigue damage under service load, the control of matrix cracking is expected to enhance fatigue life of FRP composites.
Creep behavior

Creep is defined as the strain increase of material under sustained stress over a long time. When subjected to a relatively high stress, a creep rupture may take place in the material, which should be avoided in prestressing applications. Due to the viscoelastic properties of FRPs, their creep behavior should be taken into consideration when designed to be used at a sustained stress. Authors’ previous researches on the evaluation of creep behavior of FRP and the enhancement method are introduced below.

Creep development

The relationship of strain and sustained time showed obviously different characteristics upon different levels of stress (Shi et al. 2015a). Under low stress, it was characterized by a first stage of rapid increase of strain and a second stage of long stable low rate of strain increase. Under moderate stress, creep rupture occurred after the second stage. Under high stress, an unstable strain increase in the second stage was characterized (Figure 15). The initial stage of rapid increase of strain was caused by the non-uniformity of fibers in the FRP composite, which causes continuous fracture of fibers under high stress or stress redistribution among the fibers under moderate and low level of stresses (Wang et al. 2014b).

Evaluation of creep and relaxation behavior

Creep behavior is generally evaluated for two aspects: creep rupture stress and creep strain increment rate. As shown in Figure 16, the creep rupture stresses of CFRP and AFRP are 0.7 \(f_u\) and 0.55 \(f_u\) (ACI 440.4R), respectively, making them potentially feasible for prestressing use. However, GFRP is considered unsuitable for prestressing applications, since its creep rupture stress is only 0.29 \(f_u\) (ACI 440.4R). Newly developed basalt FRPs possess a satisfactory creep rupture stress equal to 0.54 \(f_u\) (Shi et al. 2015a), which allows that competitive with CFRP and AFRP in creep behavior. In terms of 1000 h creep strain increment, CFRP performs the lowest among FRPs (Figure 17). The corresponding value of BFRP (1.5 %) is slightly higher than CFRP. The increment rates of AFRP and GFRP are relatively large (7 % and 5 %, respectively). Larger creep strain increment causes larger relaxation loss, which is adverse for prestressing members.

In terms of relaxation behavior, CFRP has only 3 % one-million-hour relaxation rate, which is the lowest among all types of FRPs. However, AFRP performs excessive relaxation rate (12.5 %), which makes it unsuitable for prestressing applications (Zou et al. 2003). The corresponding value of BFRP was predicted to be 6.7 % under initial stress of 0.5 \(f_u\) (Shi et al. 2016). This level of relaxation rate is acceptable in prestressing applications.
Creep behavior under multi field coupling

Marine environment has a negligible effect on the creep strain of FRP tendons at ambient temperature, but a high temperature up to 55 °C accelerates the creep deformation conspicuously (Shi et al. 2015b). At relatively low temperatures such as 25 or 40 °C, the creep rupture behavior of BFRP, after aging in marine environment, shows no degradation, as shown in Figure 18. The largest reduction of creep rupture stress occurs at aging temperature of 40 °C and duration of 9 w, which is only 7%. This phenomenon shows good consistency with the result of fatigue behavior in marine environment. The results demonstrate the feasibility of BFRP tendons applied as prestressing components in marine environment.

Enhancement of creep behavior

The uneven fibers in FRP can be straightened and redistributed during the pretension process because of the viscoelastic deformation of resin, thus, creep behavior can be enhanced through a two-stage pretension treatment during and after production (Figure 19). Authors’ previous research showed that pretension level of 0.6 $f_u$ and duration of 3 h is the optimized pretension condition for BFRP (Wang et al. 2015b). As shown in Figure 20, the creep strain increment rate of FRP was reduced by more than 50% through pretension treatment. Furthermore, the one million-hour creep rupture stress of the pretension-treated BFRP tendons is effectively increased from the original 0.59 $f_u$ to 0.63 $f_u$ (based on experimental fitting) and from 0.52 $f_u$ to 0.54 $f_u$ (according to the reliability analysis). The enhanced creep rupture limit promotes more sufficient use of BFRP tendons (Shi et al. 2015a).

LONG-TERM BEHAVIOR OF FRP-CONCRETE INTERFACE

Debonding of the external FRP is a main failure mode in structures strengthened by FRP. Thus, the long-term behavior of FRP-concrete interface is a key factor controlling the service life of these types of structures. In this respect, authors have done a series of studies, which are presented as follows.
Failure modes

The failure mode of FRP-concrete interface (Figure 21) depends on the loading type and level. At higher fatigue loading levels, fracture propagates along concrete layer. This failure mode is dominated by concrete damage (micro cracks) accumulation. In contrast, at low fatigue loading levels or creep load, interfacial failure develops along the adhesive, which is controlled by the adhesive creep deformation.

Modelling of long-term behavior of FRP-concrete interface

Wu and Diab (2009) proposed some modelling methods to simulate the long-term behavior of FRP-concrete interface. Time-dependent (fatigue and creep) fracture process of the FRP-concrete interfaces was divided into two stages: (1) a debonding initiation stage, which relies on a linear viscoelastic model (Figure 22 (a)) controlled by the time of creep or total number of fatigue cycles and the properties of the adhesive layer; and (2) a time-dependent debonding propagation stage which depends on a bond-slip model (Figure 22 (b)) and a damage accumulation factor.

Throughout the experimental results, the above models showed their ability to predict the debonding and crack propagation along FRP-concrete interface not only under low fatigue and sustained loadings, but also under high loadings (Figure 23). Furthermore, these models gave the opportunity to numerically study the creep and fatigue performance of structures externally strengthened with CFRP sheets taking into account the deterioration effect of FRP-concrete interface.
Degradation of FRP-concrete interface with long-term aging

Water molecules break the hydron bond and VDW of resin/SiO$_2$, leading to lower fracture energy of interface and degradation of bond behavior. As shown in Figure 24, before aging, the failure takes place in concrete, while after aging in water for 360 days, the failure occurs in the adhesive. Degradation of the interface can be clearly demonstrated from the fracture energy with respect to aging duration (Pan et al. 2015).

Enhancement of FRP-concrete interface

Polymer treatment (Figure 25) can enhance concrete layer, assuring its bonding behavior with the adhesive under long-term fatigue load. With polymer treatment, failure of FRP-concrete interface takes place in the concrete layer, rather than in the adhesive. The bond-slip curve of interface is also significantly higher than the one without polymer treatment (Figure 26). Thus, polymer treatment improves the long-term behavior of interface.

LONGEVITY DESIGN OF FRP CABLE SUPPORTED BRIDGES

In cable supported bridges, the steel cables can induce spanning limitation and decrease in safety and service life due to their large self-weight and long-term behavior deficiency. To overcome these limitations, the application of FRP cables is adopted in long-span cable supported bridges to achieve light weight, longer span and durability (Meier 1987; Wang and Wu 2010). A series of FRP cables have been developed by the authors’ team. Aside from CFRP cables, the authors’ team has addressed issues on developing BFRP as cables owing to their superior fatigue and creep behaviors as well as their high strength. Meanwhile, to overcome the shortcomings of CFRP, such as high cost, brittleness and deficiency in aerodynamic stability, hybrid FRP cables were developed to achieve the balance between cost and performance. The adoption of various FRP cables in cable-stayed bridges not only can achieve higher static performance compared to steel cable bridges, but also benefit lowering the
resonance possibility between stay cables and bridge deck (Wang and Wu 2010; Wang et al. 2013). In addition, the adoption of FRP cable can enhance the damping effect of cable and benefit vibration control, which has been demonstrated by reduced scale experiments of various FRP cables (Wang and Wu 2011a, 2011b; Yang et al. 2014, 2015). In addition, the breakthrough of anchor method of large capacity FRP cable provides more feasibility for FRP cable bridges with the assurance of high performance (Wang et al 2014a, 2015c). Those studies indicate that various FRP cables can enhance bridge behavior and realize long service life. The following introduces the concept of longevity design of bridges with FRP cables and the key compositions supporting the detailed design.

Concept of longevity design

The concept of life-controllable design of long span bridges is proposed based on the consideration of bridge span, service life requirement and economic index as shown in Figure 27. Span requirement of bridge determines the selection of different types of FRP cables, while service life requirement can be obtained by controlling design stress of FRP cable according to their fatigue and creep behavior and also bridge stiffness needs. The economic index is also required by life design due to the maintenance cost minimization, which can be realized by proper cable arrangement. The following section articulates the above three key issues.

Cable selection based on the bridge span

In order to avoid ineffective use of FRP composites in different spans of bridges, different types of FRP cables including hybrid FRP cables were recommended to be used in various spans of bridges according to their sufficient performance in both strength and stiffness. The applicable span for different kinds of cables were evaluated to be 1,200 m, 1,600 m, 1,900 m, 2,100 m, 3,500 m, 6,040 m, and 10,000 m for steel, B/SFRP 30%, B/SFRP 20%, BFRP, B/CFRP 25%, B/CFRP 50%, and CFRP cables, respectively. Based on the applicable span of different FRP cables, the superior performance to ratio selection of cables in bridge can be obtained.

Optimization of cable design stress

The design method for different FRP cables in long-span bridge is a key issue to control bridge life as shown in Figure 29. The design stress of FRP cables not only needs to take into account the structural behavior, especially the nonlinear behavior caused by cable sag effect, but it should also consider the fatigue and creep limit of each type of cable, which usually control service life of the critical areas of bridges. Based on the above considerations, the proper design stress of FRP cable can be determined by the structural analysis of the entire bridge with certain requirements in stiffness.
Determination of cable arrangement

To minimize the maintenance cost of bridges and realize LCC cost, the hybrid cable arrangement is a potential selection. The concept is to adopt different FRP cables in the applicable length so that not only the superior performance can be realized but also LCC cost can be lowered.

CONCLUSIONS

Focusing on the longevity design of major structures strengthened/reinforced with FRPs, this paper evaluated the long-term behaviors of FRP composites and FRP-concrete interface, and proposed several effective methods for their enhancement. Furthermore, by focusing on new construction with FRPs, the design methods to realize high performance and longevity of FRP cable supported bridges were introduced. The main conclusions were drawn as follows:

1. Fatigue and creep behavior of FRP composites are two key factors controlling the service life of structures subjected to the cyclic and sustained load at a high stress level. The newly developed basalt FRP composites show advantageous fatigue and creep behavior, which makes it suitable for realizing the longevity of major structures. Marine environment causes degradations in fatigue and creep life, but the reductions are limited and the structural requirements can still be fulfilled. Furthermore, the long-term behaviors of FRP composites can be effectively enhanced through hybridization, matrix modification and pretension treatment.

2. The long-term behavior of FRP-concrete interface is another key factor controlling the service life of structures strengthened by external FRPs. Through the modelling methods proposed by the authors, the creep and fatigue behavior of the interfaces were simulated accurately. Polymer treatment, which enhances concrete layer, can improve the long-term behavior of FRP-concrete interface.

3. Based on the research studies on fatigue and creep behavior of FRP composites and mechanical and economic behavior of FRP cable-stayed bridge, the concept of life-controllable design of long-span bridges is proposed based on the consideration of bridge span, service life requirement and economic measures. By these three key issues, the longevity of bridges with FRP cables is expected to be well controlled and designed.

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FRP SANDWICH STRUCTURES IN BRIDGE AND BUILDING CONSTRUCTION

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ABSTRACT

Selected applications of fiber-reinforced polymer (FRP) sandwich structural elements in building and bridge construction are discussed. It is shown how in building construction this technology enables the integration of functions such as load-bearing capacity, thermal insulation, daylight entry and energy production, into single, large-scale and lightweight roof or façade panels. In bridge construction, the high resistance against environmental impact enables conceiving much simpler construction details because no sealing layer is required, and lightweight deck panels enable the widening of existing bridges without additionally loading the substructure.

KEYWORDS

Sandwich structures, bridge construction, building construction, function integration.

INTRODUCTION

Fiber-reinforced polymer (FRP) sandwich structures offer high structural efficiency and are lightweight – the reasons why successful applications in aerospace, naval, automotive and rail industries are widespread. Due to their advantageous properties, load-bearing FRP sandwich structures are also arousing increasing interest in civil infrastructure, i.e. bridge and building construction; a growing number of pilot projects have been built in recent years.

In building construction, the potential multifunctionality provided by FRP sandwiches, i.e. the possible integration of structural, architectural, building physics and energy supply functions, is of great interest. The sandwich concept basically allows the integration of structural and building physics functions, i.e. the merging of building envelope (thermal insulation) and load-bearing structure. The flexibility and transparency of optically optimized glass-FRP (GFRP) materials further enables the integration of architectural functions (complex shapes, light, color) and energy supply functions (encapsulation of photovoltaic cells in transparent skins). Function integration results in a significant reduction in the number of building components, which can be prefabricated under controlled conditions, and thus improves quality compared to cumbersome multilayered on-site production using traditional materials such as concrete or steel.

In bridge construction, lightweight FRP sandwich decks are corrosion-free and thus allow construction details to be simplified and maintenance costs reduced. The replacement of heavy concrete decks may also allow the widening of existing bridges without increasing the total load on the substructure. In both building and bridge construction, lightweight components can be large scale but nevertheless easily transported to the construction site and rapidly installed, thus shortening construction time and compensating for higher material cost. Selected projects in Switzerland, already built or in the realization stage, demonstrate these advantages of FRP sandwich structures in civil infrastructure in the following.

FRP SANDWICH STRUCTURES IN BUILDING CONSTRUCTION

Eyecatcher Building, Basel, 1998

The five-story Eyecatcher Building in Basel, still the tallest FRP building in the world, was conceived as a reusable temporary building. After a first year of being used for a building fair at one location in Basel in 1999, it was
dismantled and reinstalled at another location in Basel where it still remains and is used as an office building, see Figure 1. The primary load-bearing structure is composed of three parallel multilayer GFRP frames connected by timber slabs. Translucent sandwich panels are used for the side-facades and were also made of GFRP. They consist of two thin face sheets separated by a sheet with trapezoidal corrugations. A fibre content of approximately 30% results in a translucency of approximately 70%. To provide thermal insulation, the panels are filled with aerogels, which can be either opaque or translucent. With a panel thickness of only 50 mm it was possible to obtain a U-value of 0.27 W/m²K (Keller et al. 2004).

**Novartis Main Gate Building, Basel, 2006**

The lightweight 400-m² GFRP web-core sandwich roof of the Novartis Main Gate Building is supported by only four glass walls, giving the building maximum transparency, see Figure 2. The sandwich construction integrates static functions (vertical load transfer and bracing of the glass walls), building physics functions (thermal insulation and integration of an acoustic ceiling) and architectural functions (double-curved wing shape), which allowed the prefabrication of the entire roof in only four panels that were easily transported to the site and rapidly installed. CNC cutting of 460 PU foam blocks up to 600-mm-thick, each of a different geometry, and adhesive bonding proved to be advantageous procedures for the fabrication of the complex shape, without the use of expensive molds (Keller et al. 2008).

**CLP Building, Montreux (Project)**

The 1000-m² GFRP web-core sandwich roof of the CLP Building, replaces a timber roof over an indoor swimming pool. It provides the inside space with architectural spatial effects thanks to double-curved enclosure shapes that bring daylight through the roof, see Figure 3. The impact of the complex shapes on the manufacturing cost is decreased by the repeated use of one (expensive) double-curved mold. The lightweight GFRP sandwich construction was selected for two main reasons: to allow the use of the existing vertical structural members as supports within their limited load-bearing capacity, and to meet demanding serviceability and durability requirements, particularly concerning performance under high temperature and relative humidity service conditions (up to approximately 34°C and 80%, respectively). The prefabrication of the roof, envisaged in the form of 5-m-wide panels transportable by road to the site, is expected to speed up the installation process (which could be done almost simultaneously with the dismantling of the existing timber roof) and thus reduce the swimming pool closure time.
A/GFRP Thermal Break Element, 2016

A multifunctional highly-insulating aramid/glass-FRP (A/GFRP) thermal break element has been developed for the transfer of moment and shear forces from an outside balcony concrete slab through the thermal insulation layer of a building envelope to the inner concrete slab. The new connector considerably improves the energy savings of buildings due to its low thermal conductivity which is 115 (20) times smaller for AFRP (GFRP) compared to the currently used stainless steel bars in such connectors. The moment-tensile force of the balcony cantilever is transferred by an AFRP loop, the compression force by a short pultruded GFRP element. The latter is connected to a hexagonal AFRP or GFRP sandwich element with polyurethane core, which transfers the shear force through an inclined compression diagonal. This combined compression-shear element is industrially fabricated by filament winding. The 55mm insulation gap, located in the insulation layer, is filled with highly insulating aerogel granulate (Goulouti et al. 2016).

FRP SANDWICH STRUCTURES IN BUILDING CONSTRUCTION

Avançon Road Bridge, Bex, 2012

The GFRP-balsa-steel bridge over the Avançon River is designed for 40 tons truck loads. The new two-lane lightweight bridge replaced a deteriorated one-lane concrete bridge without increasing the total load on the old stone abutments and foundations below the riverbanks, which could thus both be maintained despite the widening and doubling of the traffic loads, see Figure 5. The GFRP-balsa sandwich deck – of 11.45-m length, 7.50-m width, 285-mm thickness, including a skew angle of 65° and longitudinal slope of 8% – was prefabricated by vacuum-infusion in three panels. The deck consists of two 22-mm thick GFRP face sheets and a 241-mm thick balsa core, the latter composed of a laminated veneer lumber with fibers oriented perpendicularly to the face sheets. The deck panels were adhesively bonded onto two steel girders adjacent to the existing bridge. The transverse deck joints were subsequently manufactured using on-site adhesive vacuum infusion. After removal of the concrete bridge, the new bridge was installed in one day and the bridge closure was thus reduced from 50 days (required for a conventional replacement) to 10 days. The semi-integral bridge concept, without expansion joints and separate sealing (required on concrete decks), and thus entailing much simpler construction details, significantly reduces the maintenance required (Keller et al. 2014).
Figure 5 Avançon Road Bridge, Bex – GFRP-balsa sandwich deck: panel composition, panel with skew angle and asphalt adhesion layer, installation with pre-cast concrete end-beams

**TSCB Pedestrian Bridge, Bissone**

The 18-m-span pedestrian bridge is designed as a spatial frame composed of six identical combined modules – of 3-m length, 2.4-m height and up to 4.2-m width each – made of carbon-FRP (CFRP) sandwich sections, see Figure 6. Each combined module comprises two half pieces (basic modules) with the same geometrical shape – except for an additional opening in the roof – and different thicknesses, ranging from 30 to 40 mm, for the roof, wall and deck sandwich elements, which can therefore be fabricated using the same mold. The modules will be assembled at the production site, allowing the manufacturing of the vacuum-infused module-to-module joints, the project’s most challenging construction elements, under controlled conditions. The fully prefabricated, one-piece bridge will be transported to the site ready for installation on the previously built foundations and abutments. The proposed modular system also enables the construction of footbridges of different span lengths using the same mold, thus offsetting the costs associated with their complex geometry by tailoring the sandwich material properties and/or thickness for an optimized design.

Figure 6 TSCB Pedestrian Bridge, Bissone – CFRP sandwich space frame (architect F. Broggini, Bellinzona): architectural model, prototype module, architectural rendering

**FRP SANDWICH STRUCTURES - OUTLOOK**

**Photovoltaic Cell Encapsulation**

GFRP materials may be translucent or transparent if the optical refraction indices of glass fibers and matrix match. The potential transparency allows the encapsulation of photovoltaic cells in load-bearing GFRP sandwich skins and may thus add an energy supply function, see Figure 7. Replacing the traditional polycrystalline silicon cells, which are opaque, with transparent dye solar cells enables the integration of translucent energy production zones (with transparent aerogel insulation) into GFRP sandwich structures in building construction (Keller *et al.* 2010, Agullo 2014).

Figure 7 Encapsulation of photovoltaic cells into translucent GFRP sandwich skins: flexible thin-film silicone cells, sandwich with encapsulated cell under loading, translucent DSC cells encapsulated in GFRP laminate
Multifunctional Fire Resistant Components

Multifunctional components in building construction, as described above, can further include a cellular structure with cells containing slowly circulating water, as in existing underfloor heating or cooling ceiling systems, see Figure 8. The water serves as heat transfer medium for several purposes: fire resistance through storage and removal of heat, interior room heating during winter and cooling during summer, thermal mass, but also as potential absorber for integrated thermal solar systems or cooling of photovoltaic cells. Furthermore, the extension of the water system to capture geothermal energy may extend the above-mentioned energy supply function. Fire performance experiments have confirmed the efficiency of water cooling systems: fire endurances of up to two hours were obtained for cellular GFRP floor systems (Tracy 2005).

![Multifunctional wall or roof component](image)

Figure 8 Multifunctional wall or roof component

Complex Core Assemblies

The load-bearing performance of FRP sandwich structures may be significantly improved by optimizing the core material selection and composition. Combining balsa wood of different densities for example and inserting a thin CFRP arch into a slab of (maximum) 800-mm thickness enables the span of GFRP sandwich slab (road) bridges to be increased by up to 19 m, see Figure 9. Combining further GFRP sandwich decks with steel main girders and adding timber and steel inserts above the (adhesively-bonded) deck-to-girder joints to improve composite action allows the span of such lightweight hybrid GFRP-steel bridges to be increased by up to 30 m, see Figure 10 (Osei-Antwi et al. 2013, 2014).

![Concept for slab bridges of up to 19m span, complex core system consisting of balsa with integrated CFRP arch](image)

Figure 9 Concept for slab bridges of up to 19m span, complex core system consisting of balsa with integrated CFRP arch

![Concept for girder bridges up to 30m span, complex core consisting of balsa with timber and steel inserts (dimensions in mm)](image)

Figure 10 Concept for girder bridges up to 30m span, complex core consisting of balsa with timber and steel inserts (dimensions in mm)
CONCLUSIONS

Requirements regarding length of the design working life, live load level, construction within the built environment and building physics differentiate GFRP sandwich applications in bridge and building construction from other fields.

In building construction, the integration of structural, building physics and architectural functions into large-scale, prefabricated and lightweight FRP sandwiches, which can be rapidly installed, may lead to efficient and sustainable solutions of high quality.

In particular, the potential transparency of GFRP laminates and free formability may contribute to architecturally attractive freeform structures with integrated energy production through solar cells.

In bridge construction, the use of FRP sandwich decks with complex core assemblies may result in durable and economic solutions which can minimize traffic interruptions and increase the load-carrying capacity or enable the widening of existing bridges without overloading the substructure.

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BIO-BASED FRP COMPOSITES – DO THEY HAVE A PLACE IN CONSTRUCTION?

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ABSTRACT

Fibre-reinforced polymers (FRP) are increasingly being used in the field of Civil Engineering construction; however, with growing demand, they could pose an environmental concern. Conventional FRP systems are derived from non-renewable sources and require extensive energy to produce. Throughout the past several years, the authors have been conducting a series of investigations into the replacement of common FRP constituents with sustainable alternatives that include resins derived from under-valued agricultural bi-products such as corn cobs and sugar cane as well as natural fibres such as flax. Through these investigations, the short- and long-term performance of multiple FRP-combinations have been characterized. The authors have established bio-based FRP combinations which can replace FRP in common applications, such as confinement of concrete columns and flexural strengthening of beams. In addition, bio-based FRPs can extend to new construction such as the development of structural insulated panels for lightweight rapid construction. Through appropriate selection of material, the authors demonstrate that bio-based composites have a place within construction. At the same time, some challenges exist and have been identified. In these studies, the authors approached this topic from the view angle of mechanical properties characterization that relate to the expertise of a structural engineer, rather than the underlying chemistry and micro-structure analyses. Nonetheless, the latter is equally important but is beyond the scope of these studies and the expertise of the authors.

KEYWORDS

Bio-based FRP, bio-resin, furfuryl alcohol, epoxidized pine oil, natural fibres, flax fibres, durability.

INTRODUCTION

Fibre-reinforced polymers (FRP) systems are increasingly being accepted in new construction and retrofits. Their high strength-to-weight ratio, ease of installation and corrosion-resistance make them ideal for many structural applications. However, conventional FRP components are derived from petroleum and require extensive energy to produce. Therefore, they are considered an unsustainable long-term solution to society’s structural engineering demands. FRP systems are composed of fibres and a resin matrix. Conventional fibres, such as fibreglass and carbon fibre, require extensive energy to produce. For example, carbon fibre has an embodied energy of 355 MJ/kg and fibreglass has an embodied energy of 31.7 MJ/kg, whereas a natural fibre such as flax has an embodied energy of only 2.75 MJ/kg (Cicala et al. 2010).

One potential option to derive a more sustainable alternative to conventional FRP systems is to replace the components of the traditional system with more environmentally friendly alternatives. Plant-based resins may be a suitable alternative to conventional ones. These systems range from partial resin replacement via functionalized vegetable oils, such as epoxidized pine oil, to full resin replacement, such as furfuryl alcohol resin derived from corncobs and sugarcane. However, limited research has been done on these systems for structural engineering.

It is of the utmost importance to determine the short- and long-term performance of new materials and systems. This will enable engineers to provide a sustainable solution to structural demands. As a novel system, there is a large gap in information for FRP derived from natural materials. This paper summarizes the pioneering efforts of the authors in introducing bio-based composites in structural engineering applications in Canada, towards developing natural ‘green’ FRPs. The aim is to address the question of whether bio-based FRPs can meet the demands of current conventional FRP systems.
DEVELOPMENT OF NATURAL FRP SYSTEMS

The development of a natural FRP system should address both components of the FRP system, fibre and resin matrix, while balancing structural, environmental and practical reasons. To meet these demands, the authors studied the replacement of both the fibre and the resin of varying degrees.

Materials

Three resin systems and three unidirectional fibre types were studied. This includes conventionally-used constituents and natural alternatives. The laminate combinations will be denoted by a combined letter system in the order of resin, fibre and manufacturing method. For example, FP-GF-WL FRP represents an FRP system that uses furfuryl alcohol resin cured with p-Toluenesulfonic acid monohydrate (FP) and glass fibres (GF), and is manufactured using the wet lay-up moulding technique.

Resin Systems

_Epoxy (SE):_ Tyfo S, a commercial epoxy resin with a reported post-cured tensile strength and modulus at 60°C for 72 hours of 72.4 MPa and 3.18 GPa, respectively. It exhibits a maximum elongation of 5%.

_Epoxy GR (EP):_ An epoxy blend comprised of epoxidized pine oil and bisphenol A/F with a reported post-cured tensile strength and modulus at 60°C for 72 hours of 58.8 MPa and 2.63 GPa, respectively. It exhibits a maximum elongation of 2.6%.

_Furfuryl alcohol (FA):_ QuaCorr 1001, a commercial furfuryl alcohol resin which is derived from renewable resources such as sugar cane and corn cobs. It has a dark-reddish to brown color, and has the following physical characteristics: a 1.22 specific gravity, viscosity of 300-600 cps and a flash point of 75.6°C. Two catalysts were used to cure the resin: QuaCorr 2001 (FQ), a proprietary catalyst, and p-Toluenesulfonic acid monohydrate 97.5% (FP). A 3% per weight ideal concentration of either catalyst was established (Fam et al. 2013).

Fibres

_Carbon fibres (CF):_ Tyfo SCH-41, a 1.74 kg/m² unidirectional carbon fibre fabric with a reported tensile strength and modulus of 4,000 MPa and 230 GPa, respectively. It exhibits a maximum elongation of 1.7%.

_Glass fibres (GF):_ Tyfo SEH-51A, a 2.55 kg/m³ unidirectional E-glass fibre fabric with a reported tensile strength and modulus of 3,240 MPa and 72.4 GPa, respectively. It exhibits a maximum elongation of 4.5%.

_Flax fibres (FF):_ Biotex Flax UD, a 1.5 kg/m³ unidirectional flax fibre fabric with a reported tensile strength and modulus of 500 MPa and 50 GPa, respectively. It exhibits a maximum elongation of 2%.

Manufacturing Methods

_Wet Lay-up (WL) Moulding_: A manufacturing method in which the fibres are saturated by hand and cured in ambient conditions. No additional equipment or resources were used during this process. This is representative of many retrofitting methods which use dry fibres and an impregnating resin.

_Vacuum Bag (VB) Moulding_: A manufacturing method in which the FRP, after being saturated similar to that of the wet lay-up process, is subjected to a constant pressure to improve impregnation, eliminate voids and increase fibre volume fraction. A rotary vane pump provided a constant pressure of 72±2 kPa of pressure during the initial 24 hours of curing.

Characterization of Natural FRP Materials

Short-term Performance

The tensile performance of FRP composites composed of the above-mentioned resins and fibres was investigated as a preliminary study to assess the viability of natural FRP systems. A summary of the tensile tests is given in Table 1.
Two general trends can be observed from the preliminary coupon tests. Firstly, the replacement of SE with EP, FP and FQ showed minimal reduction in ultimate tensile strength and modulus of up to 10%, except for FP-GF-WL. The variation is typically within standard deviation. Secondly, the replacement of GF with FF showed a reduction of ultimate tensile strength and modulus to a third of the conventional counterpart. These two trends highlight the known fact that the fibre contributes significantly more to the mechanical properties of the composite than the resin. Furthermore, it helps outline the potential fields of application for each of the FRP composite systems based on the impact of using a bio-based replacement.

Table 1: FRP tensile coupon test matrix and results, where all values are based on an average of five repetitions (Fam et al. 2013; Mak et al. 2015a; Mak et al. 2015b; McSwiggan and Fam 2016)

<table>
<thead>
<tr>
<th>Type</th>
<th>Resin</th>
<th>Fibre</th>
<th>Manufacturing Method</th>
<th>Ultimate tensile strength (MPa)</th>
<th>Tensile modulus (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Mean</td>
<td>Stand. Dev.</td>
</tr>
<tr>
<td>SE-CF-WL</td>
<td>Epoxy</td>
<td>Carbon</td>
<td>Wet Lay-up</td>
<td>947</td>
<td>74.9</td>
</tr>
<tr>
<td>SE-GF-WL</td>
<td>Epoxy</td>
<td>Glass</td>
<td>Vacuum Bag</td>
<td>525</td>
<td>17.0</td>
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<tr>
<td>SE-GF-VB</td>
<td>Epoxy</td>
<td>Glass</td>
<td>Vacuum Bag</td>
<td>735</td>
<td>47.2</td>
</tr>
<tr>
<td>SE-FF-WL</td>
<td>Epoxy</td>
<td>Flax</td>
<td>Wet Lay-up</td>
<td>150</td>
<td>10.4</td>
</tr>
<tr>
<td>SE-FF-VB</td>
<td>Epoxy</td>
<td>Flax</td>
<td>Vacuum Bag</td>
<td>177</td>
<td>10.1</td>
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<tr>
<td>EP-CF-WL</td>
<td>Epoxidized Pine</td>
<td>Carbon</td>
<td>Wet Lay-up</td>
<td>948</td>
<td>89.9</td>
</tr>
<tr>
<td>EP-GF-WL</td>
<td>Oil Blend</td>
<td>Glass</td>
<td>Vacuum Bag</td>
<td>481</td>
<td>31.4</td>
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<td>EP-FF-WL</td>
<td>Furfuryl alcohol</td>
<td>Flax</td>
<td>Vacuum Bag</td>
<td>135</td>
<td>11.9</td>
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<tr>
<td>EP-FF-VB</td>
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<td>Flax</td>
<td>Wet Lay-up</td>
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<td>FP-CF-WL</td>
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<td>Carbon</td>
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<tr>
<td>FP-GF-WL</td>
<td>Furfuryl alcohol</td>
<td>Glass</td>
<td>Wet Lay-up</td>
<td>520</td>
<td>22.0</td>
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<tr>
<td>FQ-GF-WL</td>
<td>Furfuryl alcohol</td>
<td>Glass</td>
<td>Wet Lay-up</td>
<td>83.8</td>
<td>48.19</td>
</tr>
</tbody>
</table>

Long-term performance

To gain a better understanding of the long-term environmental performance of the FRP systems, a selection of the FRP composites were conditioned in a 3.0-3.5% salt concentration solution at three temperatures below the wet glass transition temperature, for up to 365 days. In most instances, 23, 40 and 55°C were the set temperatures.

The combination of salt solution and elevated temperatures were reported as the most suitable and efficient method to evaluate the impact of environmental conditioning at an accelerated rate (Al-Zahrani et al. 2002; Cromwell et al. 2011). Furthermore, the Arrhenius equation was used to determine the service life of FRP (Bank et al. 2003). An in-depth description of the procedure is described by Mak et al. (2015b).

The strength retention of the FRP composites are shown in Table 2. The strength retention is shown at 180 days of conditioning for all temperatures and specimens, so that all composites can be compared at the same conditions. This is not the maximum conditioning period. The difference in the maximum conditioning temperature of 50 and 55°C was to ensure that the FP resin did not post-cure in the conditioning tank. SE- and EP-equivalent variations were also conditioned at 50°C to allow for comparison.

The total conditioning period was taken into account to predict the strength retention after 100 years of service at 3°C, 10°C and 20°C. These temperatures correspond to average temperatures in central Canada, southern parts of Canada and southern parts of the United States. Predictions were not made for all FRP composites at this time.

Table 2: Strength retention after 180 days of conditioning and prediction after 100 years of service (Eldridge and Fam 2014b; Fam et al. 2013; Mak et al. 2015a; Mak et al. 2015b; McSwiggan and Fam 2016)

<table>
<thead>
<tr>
<th>Type</th>
<th>Strength retention after 180 days (%)</th>
<th>Predicted Strength Retention after 100 years (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>23°C</td>
<td>40°C</td>
</tr>
<tr>
<td>SE-CF-WL</td>
<td>99.40</td>
<td>89.30</td>
</tr>
<tr>
<td>SE-GF-WL</td>
<td>94.06</td>
<td>76.97</td>
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<td>SE-FF-WL</td>
<td>82.00</td>
<td>81.20</td>
</tr>
<tr>
<td>SE-FF-VB</td>
<td>77.70</td>
<td>64.10</td>
</tr>
<tr>
<td>EP-CF-WL</td>
<td>100.15</td>
<td>102.8</td>
</tr>
<tr>
<td>FP-CF-WL</td>
<td>85.60</td>
<td>83.65</td>
</tr>
<tr>
<td>FQ-GF-WL</td>
<td>83.83</td>
<td>48.19</td>
</tr>
</tbody>
</table>
Three major trends can be observed from long-term environmental aging. Firstly, the most prominent difference between the resin systems is that the FA resin exhibited a lower strength retention compared to alternative resin systems. Furthermore, this difference even exists between two FA systems: FP-CF-WL and FQ-GF-WL. Although these are two different fibres, it highlights a necessity to further assess the environmental resistance of the composite. Secondly, composites containing CF have a better strength retention when subjected to elevated temperature conditioning compared to GF and FF. This is likely due to the bond between the fibre and resin. Lastly, FF demonstrated a more immediate drop in strength retention; however, its strength stabilizes sooner. This is likely due to the immediate effect of moisture on the natural fibres.

Due to the different chemical structure of FA resins compared to SE resins and the lower strength retention observed above, Foruzanmehr et al. (2016) studied FQ-GF-WL in further detail in comparison to SE-GF-WL. To simulate a harsh environment condition, the FRP was subjected to an alkaline solution of up to 180 days at 60°C. This is harsher than real-life scenarios. After six months of conditioning, SE-GF-WL showed no signs of hydrolysis degradation, whereas FQ-GF-WL showed signs of hydrolysis degradation leading to the failure of the backbone of the polymer. When assessing the inter-laminar shear stress (ILSS), SE-GF-WL showed a significant decrease in capacity of 22% after six months, which is likely due to interface failure between the fibre and resin. FQ-GF-WL showed a significant decrease in capacity of 30% after six months, which is due to the degradation of the resin. SEM showed that the resin-fibre interface is better for FQ-GF-WL composites than SE-GF-WL composites, which supported the ILSS results. Furthermore, FQ-GF-WL showed a higher water absorption capacity due to its lower hydrophobicity and higher porosity. The study suggested that FA resins may exhibit some deficiencies when subjected to environmental conditioning; however, it performs better than conventional resin systems in other respects.

**Future Work**

To expand on the long-term performance of FA, the authors are in the process of investigating the mechanical performance of FQ-CF-WL and FQ-GF-WL when subjected to tension-tension cyclic/fatigue loads.

To address the challenges associated with the last two trends for FF, the authors are currently studying the effects of moisture on natural fibres, specifically wet-dry cycles and freeze-thaw cycles. In addition to SE-FF and EP-FF, the research will incorporate treatment of FF to improve the resin-fibre interface.

**APPLICATION OF A NATURAL FRP SYSTEMS**

**Resin Replacement – Retrofitting Applications**

The most common application for FRP is retrofitting and rehabilitating current structures. This is due to the high strength-to-weight ratio, ease of application and minimal effect on the member dimensions. Based on the previous section results, it was deemed that the most appropriate resin-fibre combinations for ‘heavy-loading’ structural application such as retrofitting reinforced concrete structure would be bio-based resins and conventional fibres such as glass and carbon. The tensile strength and modulus of natural fibres (flax in this case) were quite low.

**Contact-Critical Applications – Column Confinement**

To determine the feasibility of confinement with natural FRP systems, Eldridge et al. (2014a) studied the short- and long-term durability of concrete cylinders wrapped with GF. SE was used as the control resin, since it is commonly used in practice. The bio-based alternative resin system was FQ.

Prior to the confinement study, it was necessary to establish the required overlap length to develop sufficient anchorage. Fam et al. (2013) established that FQ-GF-WL lap-splices of up to 300mm did not reach the ultimate tensile capacity of the FRP. There is an increase in shear strength as the overlap increases; however, once the overlap reaches 200mm it begins to plateau at approximately 68% of the ultimate tensile capacity. This was used as a design factor for the concrete cylinder confinement study.

The confinement study focused on concrete cylinders of dimensions 152mm-by-305mm. Concrete cylinders were prepared as plain unconfined, SE-GF-WL-confined and FQ-GF-WL FRP-confined concrete cylinders. Two layers of fabric were used for confinement. Performance was based on the concrete cylinder compressive strength. Long-term performance was determined after conditioning similar to that described in the Long-term Performance section. The unconfined concrete strength was 41.9±0.79MPa.
The performance of SE and FQ systems was similar as shown in Figure 1. The short-term performance of confined cylinders was 93.9±0.53MPa with SE and 94.0±1.06 with FQ. Additionally, both systems demonstrated a similar bilinear stress-strain curve (Eldridge et al. 2014a). There was no statistical difference between the two resin systems in this short-term test. The long-term performance showed a similar response. At the highest temperature of 55°C and longest period of 300 days, there was no statistically significant difference between the two resin systems. Both systems demonstrated approximately a 28% reduction in compressive strength. This similarity was also reflected at 40°C; however, SE-GF-WL demonstrated a slightly higher strength retention at 23°C. Therefore, FQ FRP systems may be a viable alternative to conventional SE FRP systems for concrete columns confined with FRP. Figure 2 shows the failure modes based on the two resins (further details are in Eldridge et al. 2014a).

Figure 1 Percentage compressive strength retention after 300 days of conditioning at elevated temperatures

![Figure 2 Failure mode of GF-confined concrete cylinders using a) SE and b) FQ resins](image)

**Bond-Critical Applications – Flexural Retrofitting**

To determine the feasibility of bond-critical applications, McSwiggan et al. (2016) studied the short- and long-term durability of notched-concrete prisms reinforced with CF. SE was used as a control resin, since it is commonly used in practice. The natural alternative resin systems were FP and EP. FA resin cured with p-Toluenesulfonic acid monohydrate was chosen due to the apparent environmental resistance.

A preliminary study demonstrated that FA resin-based systems do not cure or adhere properly to concrete surfaces (McSwiggan et al. 2016). As such, FP-CF-WL laminates were created as separate (prefabricated) plates and then adhered to the prisms, simulating the application of commercially available pultruded CF plates.

The study focused on concrete prisms of dimensions 305mm-by-102mm-by-102mm. Concrete prisms were all notched at mid-span up to mid-depth as shown in Figure 3. Prisms were studied as notched-plain, notched with SE-CF-WL FRP and notched with FP-CF-WL FRP. Performance was quantified based on the normalized shear strength retention, where long-term performance was determined after conditioning similar to that described in the
Long-term Performance section. Normalization was done by dividing the shear strength by the square root of $f_c'$. The design concrete strength was 35MPa; however, it varied from 30 to 44MPa, depending on the exposure conditions.

![Figure 3 Bond test specimens and setups](image)

The performance of SE, EP and FP systems is shown in Figure 4. The short-term normalized shear strength was 0.683 for SE-CF-WL, 0.722 for EP-CF-WL and 0.964 for FP-CF-WL. There was no statistical difference between SE and EP, but there was a statistical difference between SE and FP. After conditioning, the normalized shear strength remained statistically insignificant between SE and EP. The same holds true between SE and FP. At the highest temperature of 50°C, there was no statistical difference between the unconditioned and conditioned specimens; however, at 40°C and 240 days, FP showed a statistical difference when compared to an unconditioned alternative. Variability was observed between degradation strengths, which may be attributed to the combined impact of increasing concrete strength and FRP degradation due to conditioning. Overall, limited degradation was observed with FP- and EP-based FRP systems, therefore the bio-based resin systems may be a viable alternative to SE in prefabricated plates bonded using conventional epoxy paste.

![Figure 4 Normalized shear strength retention after 240 days of conditioning at elevated temperatures](image)

Due to the success of the notched-concrete prism study with regard to prefabricated bio-based laminates, the authors have expanded the study to include nine full-scale reinforced concrete beams. The beams are of dimensions 2,800 mm-by-300mm-by-175mm and simulate a direct bond-critical application, where strips of FRP are adhered on the tension side of the beam as additional flexural reinforcement. FRP sheets made of SE, EP or FP resin types and CF or GF were used to reinforce the beams. Beams were either reinforced with a single or double layer of FRP.
The beams were tested symmetrically in a four-point loading configuration with a constant moment region of 500mm. One beam did not have any FRP and served as a control, with a maximum load of 84.25kN. For one layer of reinforcement, CF-based laminates showed minimal difference between different resins and application methods. EP applied with wet lay-up, FQ applied as a laminate and SE applied as a laminate demonstrated increases in ultimate capacity ranging from 33.5 to 36.6%. There was a significant decrease in capacity gain when GF was used, notably 17.7% with EP applied with wet lay-up. With two layers of reinforcement, SE-CF-WL and EP-CF-WL increased the ultimate capacity by 49 and 54%, respectively, whereas SE-GF-WL and EP-GF-WL increased the ultimate capacity by 43 and 32%, respectively. Minimal difference was observed in the ultimate capacity between the two CF-reinforced beams, whereas there was over 10% difference in ultimate capacity for GF-reinforced beams. All beams showed a primary failure of yielding followed by shear failure, except for SE-GF-WL, which had a secondary failure of concrete crushing. Throughout these tests, it was observed that the primary factor for the performance of the reinforced beams was the type of the fibres. The impact of changing the type of resins has very minimal impact on the performance of the reinforcement.

**Future Work**

The authors are currently extending the full-scale study to incorporate freeze-thaw durability, where full-scale beams will be subjected to 300 freeze-thaw cycles to simulate conditions experienced by bridges in Northern climates. The authors are also exploring other types of partially-bio resins for direct bonding of FRP to concrete.

**Fibre Replacement – Sandwich Panels**

Given the significantly lower tensile strength and modulus of natural fibres (flax in this case), the authors have concluded that they are most suited for systems subjected to ‘light-loading’, particularly those whose failure modes are not governed by tension failure of the FRP. An example is sandwich panels with FRP skins and foam cores. These panels can be used for flooring, roofing, cladding or decking of pedestrian bridges. Their failure modes are typically governed by compression skin wrinkling or crushing or by core shear failure. As such, the authors focused on sandwich panels with flax-FRP skins.

**Flexural Performance**

To determine the feasibility of natural FRP-skinned sandwich panels, Mak et al. (2015a) studied the flexural behaviour of sandwich panels consisting of a FRP-based structural skin and a 64kg/m³ polyisocyanurate (PIR) foam core. SE-GF-WL FRP was the control structural skin due to its common acceptance in industry. FF was studied as a replacement to GF. The number of layers of FF was also studied to determine the impact of more fabric. Sandwich panels were 150mm wide and 75mm thick with a span of 900mm. Panels were tested symmetrically in four-point bending with a constant moment region of 150mm as shown in Figure 5(a).

Figure 6 shows the load-deflection responses of the FF-reinforced sandwich panels with one, three and five skin layers, compared to control GF-reinforced sandwich panel with one layer skin. For SE-FF-WL skinned sandwich panels, it was found that the optimum number of FF layers to provide a comparable response to GF, in terms of stiffness and strength, was three layers. This represents an FF skin with a total thickness that is 80% larger than the GF skin thickness. A minimal improvement in mechanical properties was observed with five layers of FF, whereas a significant reduction in performance was observed with one layer of FF. For manufacturing methods, VB sandwich panels typically yielded a lower structural capacity. Wrinkling of the skin suggests that this may be due to the reduced moment of inertia of the thinner VB skin, thereby resulting in skin instability.

To further develop this application, the authors investigated sandwich panels with varying PIR densities of 32, 64 and 96 kg/m³. A single resin type, SE, and manufacturing method, WL, were used for this study. The same test configuration was used as before, except that the foam width and thickness were 100 mm-by-50 mm. FF-skins were manufactured using three layers.

A general trend was observed where increased core density resulted in increased flexural performance. For SE-FF-WL, increasing the core density from 32 to 64 and 96kg/m³ resulted in an increased flexural capacity of 82 and 213%, respectively. It also corresponded to an increased flexural stiffness of 65 and 150%, respectively. A similar increase was observed for GF, where flexural capacity increased by 90 and 232%, respectively, and flexural stiffness increased by 72 and 164%, respectively. No major difference was observed between the two skin types; however, the foam core density greatly impacted the failure mode, where 32 and 64kg/m³ were governed by inward skin wrinkling in the shear span and 96 kg/m³ was governed by core shear failure.
In an attempt to replace both fibres and resin, Mak et al. (2015a) studied sandwich panels of the same nature as the flexural sandwich panel study with flax fibres; however, SE was replaced with EP. EP-FF-WL sandwich panels showed a decrease in strength and stiffness of up to 23%. The decrease in stiffness is prevalent at higher loads and as such, is not noticeable within the serviceability limits of L/180 and L/360. Thus the use of a bio-based resin system in conjunction with a natural fibre is a viable option for a natural FRP given the appropriate structural system.

**Axial Loading Performance**

The authors investigated the axial performance of SE-FF-WL skinned sandwich panels with slenderness ratios ranging from 25 to 60. SE-GF-WL FRP was the control structural skin due to its common acceptance in industry. FF was studied as a replacement to GF. The number of layers of FF was also studied to determine the impact of more fabric. Sandwich panels were 100mm wide and 50mm thick and of varying heights.

The load-slenderness ratio is shown in Figure 7 for the three different core densities. Slenderness ratio is indicative of the potential for Euler buckling. For low density foams, the axial strength appears somewhat stable and independent from slenderness ratio, whereas at higher densities, a clear reduction in strength appears as slenderness ratio increases. This is attributed to a range of failure modes that was observed. By increasing the core density from 32 to 64 kg/m$^3$, the average peak load increased by 73, 56 and 72% for one, three and five skin layers, respectively. By increasing the core density from 32 to 96 kg/m$^3$, the average peak load increased by 116, 130 and 176%, respectively. Nevertheless, similar to the flexural performance, three layers of FF respond similarly to a single layer of GF for all slenderness ratios.

Failure mode of FF panels is heavily governed by slenderness. Panels with slenderness ratios of 51 and above experience global buckling at peak loads, while those with slenderness ratios below 32 experienced localized failures. The intermediate range saw a combination of global and local failure modes. The most common secondary
failure mode of panels failing primarily by global buckling is skin buckling on the concave side, especially for thicker skins. Thinner skins mostly crushed after global buckling. In some cases, both secondary failures were associated with a core shear failure, especially at high density cores. The primary local failure was outward skin buckling. This was followed by core shear failure associated with convex side skin debonding.

Figure 7 Load-slenderness plot sandwich panels under axial load for three PIR densities, where FFRP represents flax-based FRP with SE resin and GFRP represents glass-based FRP with SE resin.

**Future Work**

The authors are in the process of investigating the fatigue performance of FF-skinned sandwich panels and their performance in different environmental conditions.

**CONCLUSION**

Conventional FRP systems have become an essential part of many forms of construction; however, it poses a significant environmental impact. The authors investigated the application of bio-based resins and natural fibres as a replacement to conventional FRP constituents. Characterization of the mechanical properties of the natural FRP systems have shown both strengths and limitations, both in the natural FRP systems as well as the conventional FRP systems. As a designer, it is important to capitalize on this knowledge and design with the appropriate materials. The authors are of the belief that a combination of conventional fibres and bio-resins is suitable for ‘heavy’ structural applications, including structural retrofitting such as reinforced concrete column wrapping, while natural fibres such as flax are more suited for ‘lighter’ structural applications such as sandwich panels for cladding, roofing and decking. The authors believe that with the growing interest in sustainability and the potential shown in these systems, natural FRPs are a viable alternative to conventional FRPs in specific structural applications.

**ACKNOWLEDGEMENTS**

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ABSTRACT

The properties of Carbon Fibre Reinforced Polymers (CFRPs), very high strength, light weight, zero corrosion and no fatigue can fully be exploited for cable structures. The ideal structures for CFRP cables are highly pre-tensioned cable systems that are loaded orthogonally to their cable axes. High performance structures, such as stress-ribbon bridges, long-span roofs and facades made of CFRP can already be built economically today. To prove this point, CFRP prototype structures, a stress-ribbon bridge and a small spoked wheel cable roof built at the Technische Universität Berlin, are presented. First, however, cable structures, CFRP materials and existing CFRP cable structures are introduced in general.

KEYWORDS
Lightweight, bridges, roofs, CFRP and cables.

INTRODUCTION

A cable structure can be defined as structures in which a cable or a system of cables is used as the primary load-bearing structural element (Krishna 1978). The materials for cables have evolved from natural fibres in ancient times to wrought iron and finally to high-strength steel today (Schlaich et al. 2015).

A milestone in the development of cable structures is the Munich Olympic Stadium roof completed in 1972, which shows that the structural ideal of “leicht und weit” can be realised by using cables (Schlaich et al. 2003). Due to their ability of achieving long spans with minimal use of material while expressing lightness and elegance at the same time, cable structures are modern. Their structural forms are many and varied.

Reviewing the history of cable structures, it can be found that the development of cable materials promoted the development of new structural types. The availability of high-strength steel cables allows not only the construction of long-span cable structures, such as modern suspension and cable-stayed bridges, but also the reality of cable structures with new forms, such as cable roofs and facades. Carbon Fibre Reinforced Polymer (CFRP) is a new advanced composite material with advantages of high strength, lightweight, no corrosion and high fatigue resistance, which makes it suitable to be made into cables and potentially replace steel cables (Liu 2015).

CFRP AND CFRP CABLES

Carbon Fibre Reinforced Polymer

Carbon Fibre Reinforced Polymer, abbreviated to CFRP, is a non-mental composite material with outstanding properties. As its name suggests, it is composed of carbon fibres as the reinforcement embedded in a polymer resin as the matrix (Bhargava 2004).

The carbon fibres are very thin filaments (about 5 μm – 10 μm in diameter) containing between 90 and 100 weight % carbon (Donnet 1998). The mechanical properties of three typical carbon fibres are listed in Table 1, compared with two commonly used steel materials (Morgan 2005; Eurocode 3 2005; Eurocode 3 2006).

As shown in Table 1, the tensile strengths of carbon fibres are significantly higher than those of steel materials, while their densities are much lower. The breaking length $L_b$ (defined as the length of material hangs vertically at which it will break through its own weight) is a good parameter to show the high strength and lightweight characteristics of certain materials. It is calculated by $\sigma_u/\rho g$, where $g$ is the standard gravity constant of 9.8 m/s$^2$. As seen from the above table, the breaking lengths of carbon fibres are orders of magnitude larger than those of steel materials (Schlaich et al. 2015).
Table 1 Mechanical properties of carbon fibres compared with steel materials

<table>
<thead>
<tr>
<th>Material type</th>
<th>Density $\rho$ (kg/m$^3$)</th>
<th>Tensile strength $\sigma_u$ (GPa)</th>
<th>Young's modulus $E$ (GPa)</th>
<th>Breaking length $L_b$ (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon fibre</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Standard</td>
<td>1760</td>
<td>3.53</td>
<td>230</td>
<td>205</td>
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<td>High strength</td>
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<td>396</td>
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<td>S355</td>
<td>7850</td>
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<tr>
<td>Steel</td>
<td>Round wire</td>
<td>7850</td>
<td>1.77</td>
<td>210</td>
</tr>
</tbody>
</table>

In addition to high strength and low weight, CFRP has better corrosion and fatigue resistance and lower thermal expansion than steel. Because carbon fibres have excellent creep resistance, the stress relaxation of CFRP is very small, and thus restraining sustained stress is unnecessary for CFRP (Morgan 2005).

The above merits make CFRP an excellent material in structural engineering. The first practical utilisation of CFRPs in construction was in 1991 for strengthening the Ibach Bridge in Lucerne, Switzerland (Meier 1992). From then on, more and more CFRP products have been used not only in strengthening, repairing, reinforcing and pre-stressing but also as cables in cable structures.

**CFRP cables**

Usually, unidirectional CFRP materials are used to manufacture CFRP cables. The mechanical properties along the fibre direction of CFRPs, such as the tensile strength $\sigma_u$ and the elastic modulus $E$, are usually approximately 60% - 70% of those of the carbon fibres because the fibre volume fraction is usually 60% - 70%. It should also be noted that $\sigma_u$ and $E$ of CFRP cables are slightly smaller than those of the corresponding CFRP; this is similar to the fact that $\sigma_u$ and $E$ of steel cables are slightly smaller than those of steel wires (Liu et al. 2015).

According to their cable form and cross section, today’s CFRP cables can be classified as shown in Figure 1 (Schlaich et al. 2012). Figure 1(a) shows a CFRP cable in the form of lamella, which can be fabricated by pultrusion or lamination. Figure 1(b) shows a CFRP cable in the form of rod, which is usually fabricated by pultrusion; such CFRP cable can be made up of a single rod or a rod bundle, and the CFRP rod can be circular or deformed. Figure 1(c) shows a CFRP cable in the form of strip-loop, which is fabricated by winding a continuous CFRP strip on two pins; the strip-loop can be laminated or non-laminated. Figure 1(d) shows a CFRP cable in the form of wire rope, which is fabricated by twisting several CFRP wires into a helix; CFRP wires used are usually produced by pull-winding (Liu et al. 2015).

![Figure 1 Schematic diagrams of four types of CFRP cables](image)

The mechanical properties of four CFRP cables (in different forms and from different producers) are compared with that of a steel full locked coil rope cable in Table 2 (Winistoefer 1999; Schober and Rautenstrauch 2005; Grace et al. 2002; Grace et al. 2003; Pfeifer 2011).

As seen from Table 2, the tensile strengths of CFRP cables are higher than that of the steel cable while their densities are only approximately 1/5 of the steel cable’s density. However, CFRP cables also have some...
disadvantages, such as lower elastic modulus than that of steel cables (see Table 2), difficult anchoring and relatively high cost, which may hinder the application of CFRP cables (Liu et al. 2015).

Table 2 Mechanical properties of CFRP cables compared with steel cable

<table>
<thead>
<tr>
<th>Cable name</th>
<th>Structural form</th>
<th>Description</th>
<th>Density $\rho$ (kg/m$^3$)</th>
<th>Tensile strength $\sigma_u$ (GPa)</th>
<th>Elastic modulus $E$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DPP CFRP lamella</td>
<td>Pultruded CFRP lamella</td>
<td></td>
<td>1600</td>
<td>2.5</td>
<td>140</td>
</tr>
<tr>
<td>EMPA CFRP strip-loop</td>
<td>Non-laminated looped CFRP</td>
<td></td>
<td>1500</td>
<td>2.0</td>
<td>120</td>
</tr>
<tr>
<td>Mitsubishi Leadline</td>
<td>Parallel CFRP deformed rods</td>
<td></td>
<td>1600</td>
<td>2.3</td>
<td>147</td>
</tr>
<tr>
<td>Tokyo Rope CFCC</td>
<td>Twisted CFRP round wires</td>
<td></td>
<td>1500</td>
<td>2.1</td>
<td>137</td>
</tr>
<tr>
<td>Steel full locked coil rope</td>
<td>Twisted steel round and z-profile wires</td>
<td></td>
<td>7850</td>
<td>1.5</td>
<td>160</td>
</tr>
</tbody>
</table>

EXISTING CFRP CABLE STRUCTURES: OVERVIEW

Studies on CFRP cables in cable structures can be dated back to the early 1980s (Meier 2012). As early as in 1987, a CFRP cable-stayed bridge with a main span of 8400 m crossing the Strait of Gibraltar was proposed (Meier 1987). However, the first practical use of CFRP cables in a real cable structure was in 1996 (Karbhari 1998). From then to now, there have been eleven CFRP cable structures over the world, even though all of them were built more or less experimentally. Nine of them are listed in chronological order of completion in Figure 2 (Schlaich et al. 2014), while another two from the Technische Universität Berlin will be introduced in following sections.

![Figure 2 Nine CFRP cable structures](image)

Brief introductions of the CFRP cable structures in Figure 2 are listed as follows.
1: Tsukuba FRP Bridge; pedestrian cable-stayed bridge; all stay cables are made of CFRP; completed in 1996 and located in Tsukuba, Japan (photo credit: I. Sasaki).

2: Stork Bridge; highway cable-stayed bridge; two stay cables are made of CFRP; completed in 1996 and located in Winterthur, Switzerland (photo credit: EMPA).

3: Neigles CFRP Footbridge; pedestrian suspension bridge; two main cables are made of CFRP; completed in 1998 and located in Fribourg, Switzerland (photo credit: Tokyo Rope).

4: Herning CFRP Bridge; pedestrian cable-stayed bridge; all stay cables are made of CFRP; completed in 1999 and located in Herning, Denmark (photo credit: COWI).

5: Laroin CFRP Footbridge; pedestrian cable-stayed bridge; 16 stay cables are made of CFRP; completed in 2002 and located in Laroin, France (photo credit: Freyssinet).

6: Jiangsu University CFRP Footbridge; pedestrian cable-stayed bridge; all stay cables are made of CFRP; completed in 2005 and located in Zhenjiang, China (photo credit: K. Mei).

7: Penobscot Narrows Bridge; highway cable-stayed bridge; six strands are made of CFRP; completed in 2006 and located in Penobscot, Maine, USA (photo credit: MOT).

8: EMPA Bowstring Arch Footbridge; pedestrian bowstring arch bridge; all bowstrings are made of CFRP; completed in 2007 and located in Dübendorf, Switzerland (photo credit: U. Meier).

9: Cuenca Stress-Ribbon Footbridge; pedestrian stress-ribbon bridge; all stress-ribbons are made of CFRP; completed in 2011 and located in Cuenca, Spain (photo credit: J. R. Lopez).

CFRP STRESS-RIBBON BRIDGE

The department of Conceptual and Structural Design, Technische Universität Berlin (TU-Berlin), started research on CFRP cable structure in 2005. In May 2007, the first CFRP stress-ribbon bridge in the world was completed at the laboratory of TU-Berlin. It is still standing. After completion, an active vibration control system was installed in the bridge to reduce pedestrian-induced oscillations (Bleicher et al. 2012). The photo and sketch of this bridge are shown in Figure 3 (Liu et al. 2015). The TU-Berlin CFRP Stress-Ribbon Footbridge is a single span stress-ribbon bridge with the sag-to-span ratio of 1/60. Six CFRP ribbons are used as load bearing components in this bridge, which were provided by Carbo-Link GmbH, Switzerland. In the transverse direction, each ribbon has $2 \times 5$ layers and the total thickness is approximately 1 mm. The load bearing capacity of single ribbon is 105 kN (Schlaich and Bleicher 2007).

![Figure 3 TU-Berlin CFRP Stress-Ribbon Footbridge (a) photo (photo credit: A. Bleicher) (b) sketch](image-url)

The CFRP ribbon used in this bridge is a non-laminated pin-loaded strip-loop (see Figure 3). Such cables are manufactured by winding a very thin (approximately 0.1 mm thick) CFRP strip around two round pins (the pins are made of steel in this case) continuously. After winding, only the part between the strip start and strip end is bonded together through fusion to form a closed loop (thermoplastics matrix is adopted to produce the CFRP strip). Compared with the laminated strip-loop, the shear and radial stresses at the anchorage zone (especially the layer contacting the pin) of the non-laminated strip-loop will distribute more uniformly, because the different layers of the non-laminated one can mutually slide, thus achieving a more uniform strain distribution than the laminated one. This will lead to relatively small stress peak in the CFRP non-laminated pin-loaded strip-loop cables and help increase their ultimate bearing capacity. The cable anchorage is illustrated in Figure 4 (Liu 2015).
The above anchorage system mainly consists of two round pins, a triangular steel box and two steel anchoring bolts to connect to the ground. The anchor pin, with a diameter of 80 mm, is for holding the strip-loop, while the steering pin, with a diameter of 100 mm, is for changing the direction of the loop to facilitate tensioning and anchoring (Liu 2015).

**CFRP CABLE ROOFS AND FACADES**

All the existing CFRP cable structures, introduced in the above sections, are cable bridges. Moreover, the majority of them are cable-stayed bridges. However, cable roofs and facades are ideal structures for CFRP cables, because most of them are orthogonally loaded by external loads.

In orthogonally loaded cable structures, such as stress-ribbon bridges, cable roofs and cable facades, the structural stiffness is mainly comprised of the geometric stiffness which is induced by the pre-tension force in the cables. The elastic stiffness caused by the elastic modulus of cables is less important. For such cable structures, increasing the tensile strength of cables to increase the pre-tension force is a more efficient way to raise the structural stiffness than increasing the elastic modulus of cables. Using CFRP cables, whose tensile strength is considerably greater than that of steel cables (see Table 2), in orthogonally loaded cable structures can improve the economic efficiency, although the elastic modulus of CFRP cables is smaller than that of steel cables (see Table 2) and their unit price is also much higher. Therefore, using CFRP cables in cable roofs and cable facades will be an important component of future buildings (Schlaich et al. 2014; Liu et al. 2015; Schlaich et al. 2015).

To investigate the feasibility of applying CFRP cables to cable roofs and cable facades, a small CFRP spoked wheel cable roof was built at TU Berlin in 2013, as shown in Figure 5 and Figure 6.
In this prototype, the radial cables are loop-shaped CFRP tension members and the tension ring is a closed octagon CFRP loop. The cross section of the radial cable and the tension ring is 30 mm × 1.2 mm, which can bear nearly 80 kN tension force. Both types of CFRP structural elements were manufactured at TU Berlin by hand through laminating a continuous carbon fibre tow coated with epoxy resin on a rotating form and using vacuum technique to harden. The material of the compression ring, pillars and nodes is aluminium. To pre-tension the CFRP cables, each node of the tension ring was separated into two semilunar parts, which are linked together by four bolts. By tightening the bolts and hence shortening the distance between these two parts, the CFRP cable system of the spoked wheel cable roof was pre-tensioned to a sufficient level and a very stiff system was achieved (Liu et al. 2015).

In addition to the CFRP cable roofs, CFRP cable facades also have a bright prospect of application. Perhaps the highly pre-tensioned steel cable net facade like the one shown in Figure 7 can be made of CFRP cables in the future.
CONCLUSION AND PROSPECT

The history of building is also a history of building materials. The emergence of a new material usually leads to new structural types. CFRP could now drive the progress of cable structures.

This paper mainly reviews the existing CFRP cable structures in the world and proposes future application for this promising material. Admittedly, there are still several challenges that may hinder the wide use of CFRP cables, such as difficulty in anchoring, fire safety problems and relatively high cost. However, this should not prevent us from going ahead in this domain (Liu et al. 2015). For example, the anchorage problems could be eased by applying the CFRP Continuous Band Winding System (see Figure 8) (Liu et al. 2015). The structural engineering community can be looking forward to most interesting developments in the field of CFRP cable structures.

Figure 8 Cable roof with CFRP Continuous Band Winding System

REFERENCES


**Note**

With permission of IABSE this paper is a reprint of the following article:

REDUCING THE COST OF FRP RC RESEARCH AND DEVELOPMENT

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School of the Civil, Environmental and Mining Engineering, The University of Adelaide, Australia

ABSTRACT

Over the last few decades, the main thrust of steel reinforced concrete research and development has been based on empirical testing. While material properties may be determined empirically using relatively inexpensive approaches, semi-empirical mechanical models, which are commonly used to develop design rules, require member level testing. Hence any advancement, such as a new approach or new product, has to be accompanied by significant quantities of member level tests, which due to their scale, are both costly and time consuming. Furthermore, being empirically based, the new approach or product can and should only be used within the bounds of the experimental tests from which it was developed, which in itself places significant restriction on application. The advent of FRP reinforcement, FRP confinement, FRP retrofitting and textile reinforced concrete has radically changed the possibilities of design with reinforced concrete. If we keep to the same semi-empirical approach, as we all have through our own member testing, it will be extremely costly and time consuming to develop these advances to the stage of application and refine them further.

An alternative approach is to develop mechanics based solutions to replace the semi-empirical approaches, so that only material properties need to be determined empirically. Hence, in theory, a manufacturer of a new product, for example a new type of fibre, would only need to test for the material properties which can then be used in mechanics models for the behaviour of a member. The manufacturer can then modify the product at little expense as only material properties are required until the required member properties, obtained through the mechanics models, are achieved. In short, the manufacturer only needs to spend money on inexpensive material testing and not on expensive member testing.

This presentation explains the partial interaction material properties and partial interaction local mechanisms that the mechanics model has to incorporate and simulate in order to quantify the total behaviour of a reinforced concrete structure. This is followed by the categorisation of the different levels of research sophistication which explains the huge amount of testing currently required and the need for mechanics models. Examples of partial interaction mechanics approaches are then illustrated for quantifying the flexural rigidity at serviceability allowing for time effects. Finally a brief discussion of how advanced mechanics based design techniques will enhance the use of FRP using moment redistribution and member debonding as examples.

KEYWORDS

FRP, reinforced concrete; mechanics; partial interaction; shear; flexure; axial load; RC research; RC development; and RC design.

INTRODUCTION

Structural engineers may wish to develop for general use a new FRP product such as an FRP reinforcing bar or fibre concrete or a new type of RC FRP beam or slab or column. Or they may wish to develop a new FRP retrofitting technique for general use or simply expand the use of an existing FRP retrofitting technique or make it more accurate. Or they may simply want to develop a unique one off FRP RC member for a specific use or enhance the strength of an individual member beyond the scope of existing FRP techniques. To do this, the structural engineer has to ensure that the product or member or technique can resist a very wide range of stress resultants for members of virtually any shape and size, notwithstanding the time dependent behaviour at serviceability and the ductility required for members in a frame. Currently this requirement is done through a very large amount of member testing, the cost of which is incredibly prohibitive (Oehlers 2010; Oehlers et al 2011a; Oehlers et al 2016b; Oehlers et al 2012; Oehlers et al 2014b; Oehlers et al 2016b). However, if the structural engineer had a numerical mechanics model that can directly simulate behaviours that occur in RC members (Oehlers et al at 2016a; Oehlers et al 2016c; Oehlers et al 2014a; Zhang et al 2016a), then this would obviate the
need for member testing which would considerably reduce the cost of development. That is the engineer requires an approach which simulates the slip or partial interaction which occurs across interfaces. Even better, if the structural engineer had a mechanics analytical equation that allowed for these PI localised mechanisms (Lucas et al 2011; Visintin et al 2013a; Visintin et al 2013b; Visintin et al 2016a; Visintin and Oehlers 2016b; Zhang et al 2016b) then this would directly provide design rules for new products or members further reducing the cost of development. In this case all that is needed through testing are the material properties which once obtained do not have to be repeated.

This paper first briefly clarifies the partial interaction (PI) material properties that lead to the PI localised mechanisms that have to be simulated if a model is to directly simulate what is seen and measured in practice. Then explains why currently a large amount of member testing is required through a discussion of the different approaches to research and also shows how a mechanics based approach can save considerable costs. This is then illustrated for the serviceability behaviour of FRP RC beams and for the moment redistribution at ultimate. Furthermore, these examples will be used to show how mechanics based models that incorporate the PI localised RC mechanisms can be used to help empiricalists in: planning tests to extricate the material properties required for the design equations for the new products; and used in the inverse mode to speed up the development of new products.

RC PARTIAL INTERACTION

Figures 1 to 3 show the apparently myriad characteristics associated with RC members at all stages of loading and of which structural engineers are familiar with. All of these characteristics have to be simulated if a model is to truly represent what actually occurs in practice and, thereby, quantify strengths and deformations for all stress resultants. The task may appear daunting but these myriad behaviours can be categorized into just a few PI localised mechanisms which themselves can be simulated with even fewer PI material properties. Partial interaction (PI) being the behaviour associated with slip across an interface.

Figure 1 Characteristics of an RC beam predominant under flexure

Figure 2 Characteristics of an RC beam under shear

Figure 3 Encased column

Figure 4 illustrates a portion of an RC beam predominantly in flexure which could be prestressed or not and in which the reinforcement could be FRP or steel or a combination. The observer may see on loading the formation of near vertical flexural cracks. Their formation is governed by the PI local mechanism of tension stiffening which controls the crack spacing and crack width and consequently the flexural rigidity of the member. The observer may also see plate separation which is governed by the local PI mechanism of intermediate crack (IC) debonding which controls member debonding and consequently the effectiveness of the retrofitting technique. The observer may then see the formation of a double sliding wedge which is governed by the PI mechanism of shear sliding which controls concrete softening and consequently the ductility of the member.

For RC beams predominantly in shear as in Figure 5, the above flexural mechanisms still exist but the observer may also see the formation of a diagonal crack which forms a single sliding wedge. The force in the reinforcement, such as the inclined externally bonded plates or internal stirrups, crossing the diagonal crack is still governed by tension stiffening. However this tensile force in the reinforcement is balanced by compression in the concrete across the interface which is confinement. Hence this PI shear sliding mechanism controls the beneficial effect of confinement as well as the shear capacity of the member.
Depending on the magnitude of the stress resultants, RC columns may exhibit the localised mechanisms described above in Figures 4 and 5. Those columns with large axial loads as in Figure 6 may form a single diagonal crack. In this case the outer tube and inner stirrups provide beneficial confinement to the concrete through the PI shear sliding mechanism.

It has been shown in Figures 4 to 6, that the many characteristics associated with the behaviours of RC members in Figures 1 to 3 are governed by only three PI local mechanisms that is: (1) tension stiffening; (2) shear sliding, and (3) IC debonding. These three PI local mechanisms are due to just two PI material properties that is: (1) the PI shear friction material properties across a potential sliding plane as in Figure 7 that are often presented in the form shown in Figure 8; and (2) the PI bond-slip material properties as illustrated in Figure 9.

Hence, Figures 8 and 9 represent the only additional material testing required to simulate the PI local mechanisms shown in Figures 4 to 6 in order to form the apparently myriad characteristics shown in Figures 1 to 3. For a new product, this additional material testing is relatively inexpensive, compared with member testing, which should
mean that the development of new products should be inexpensive. Why this is not the case is explained in the following section.

![Figure 9 PI bond-slip material properties](image)

**LEVELS OF RC RESEARCH SOPHISTICATION**

The different levels of research sophistication are listed in Column 1 in Table 1 with a brief description in Column 2 (Oehlers et al 2016b). The costs are listed in Columns 3 to 5. As the cost of determining the material properties from tests applies to all levels of research this cost has not been included in the table. Where the research approach requires member testing this is a cost in Column 3. Where the research approach requires further effort in developing design rules this is a further cost in Column 4. The range of application has been included as a cost in Column 5 as those research levels with finite ranges incur an additional cost should the designer wish to go beyond the range.

<table>
<thead>
<tr>
<th>MODELS</th>
<th>COSTS</th>
<th>LEVELS OF RC RESEARCH SOPHISTICATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Name</td>
<td>Description</td>
<td></td>
</tr>
<tr>
<td>Level 7: Closed form</td>
<td>Provides fundamental understanding to structural engineers</td>
<td>Member testing: No, Design rule development: No, Range of application: Unlimited</td>
</tr>
<tr>
<td>Level 6: Simplified closed form</td>
<td>Level 7 mechanics simplified to find solutions</td>
<td>Member testing: No, Design rule development: No, Range of application: Limited</td>
</tr>
<tr>
<td>Level 5: Semi-mechanical</td>
<td>Unlimited parametric study using Level 4 results</td>
<td>Member testing: No, Design rule development: No, Range of application: Unlimited</td>
</tr>
<tr>
<td>Level 4: Numerical model</td>
<td>Provides a one-off result. Replaces experimental testing</td>
<td>Member testing: No, Design rule development: No, Range of application: Unlimited</td>
</tr>
<tr>
<td>Level 3: Simplified numerical</td>
<td>At least one parameter in numerical model determined by tests</td>
<td>Member testing: Yes, Design rule development: Yes, Range of application: Limited</td>
</tr>
<tr>
<td>Level 2: Semi-empirical</td>
<td>At least one parameter in mechanics model determined by tests</td>
<td>Member testing: Yes, Design rule development: Yes, Range of application: Limited</td>
</tr>
<tr>
<td>Level 1: Empirical</td>
<td>All parameters determined experimentally</td>
<td>Member testing: Yes, Design rule development: Yes, Range of application: Limited</td>
</tr>
</tbody>
</table>

The lowest level of research sophistication in Table 1 is Level 1: Empirical. All parameters are determined experimentally. Hence there is a cost for member testing, a cost for the development of design rules and the application is limited to within the range of the specimen properties in the member testing. This is a very useful approach at the start of the development of a new product that is to get the product on the market. The manufacturer may wish to expand the use of the product so then uses the semi-empirical approach of Level 2 where a mechanics model is used but at least one parameter of the model requires to be determined experimentally. An example is the use of full-interaction (FI) flexural rigidities that are then calibrated through tests to allow for PI. Hence there is a cost for member testing and design rule development and a limit from the range of tests to calibrate the mode. Level 3: Simplified Numerical could for example be a strain-based finite element analysis in an attempt to eliminate member testing. Being strain based it cannot cope directly with PI that is with interface slip so PI such as for tension stiffening is allowed for by adjusting the strain based material properties which is often done through calibration with test results. Hence this approach is also limited by the range of tests used in calibration. Generally speaking, Levels 1 to 3 do not attempt to simulate the PI localised mechanisms directly.

Level 4: Numerical Mechanical model in Table 1 is a numerical model that can simulate all the PI localised mechanisms in Figs. 4 to 6. As the numerical model can simulate all the behaviours and characteristics of RC members for any size and shape of member there is no need for member testing. All that is required are the material
properties for the numerical model. This model can be used to replace member testing in Levels 1 to 3 with the advantage that numerical modelling is much less expensive than member testing. Furthermore, whereas experimental testing is limited by the capacities of the laboratory there is no limit in numerical modelling to the size of the specimen being modelled. The Numerical Mechanics model can be used to provide data for a parametric study leading to Level 5: Semi-Mechanical approaches for design.

Numerical modelling, as with experimental testing, does not give the mechanics directly and as such provides the structural engineer with a shallow understanding of the behaviour to make decisions. Hence the next level of sophistication that is Levels 6 and 7 in Table 1 are closed form solutions that provide the mechanics equation that incorporates all the relevant PI localised mechanisms. When the solution provides an equation that can be used directly in design then this is a Level 7: Closed Form. If this is available then the structural engineer can use this directly in the design of a new product or member and all that is needed are the material properties. Hence the costs are minimal their being no member testing, no design development and no limit to its application. Should the mechanics be too complex to provide a direct solution, then the mechanics can be simplified to produce a Level 6: Simplified Closed Form solution such that the application should be limited to within the bounds of the simplification should there be any. The use of Levels 4 to 7 models in the development of design rules for new products that is without the need for member testing is illustrated in the following section for serviceability. This is followed by a discussing of how Levels 6 and 7 models will allow structural engineers to design for specific conditions that fall outside the scope of codes.

**TIME DEPENDENT SERVICEABILITY BEHAVIOUR OF RC BEAMS**

Consider the time dependent serviceability behaviour of RC (Sturm et al 2016; Visintin et al 2016c).

**Levels 4 and 5 Models**

A segmental analysis of a segment of a beam between two adjacent cracks in Figure 1 is shown in Figure 10(b). After flexural cracking, the tension reinforcement is simulated with a tension stiffening prism as in Figure 10(a). The segment is subjected to a shrinkage strain \( \varepsilon_{sh} \) such that if the concrete was unrestrained the concrete segment end would move from A-A to B-B in Figure 10(c). An Euler-Bernoulli deformation C-C is applied. The force in the compression reinforcement \( P_{rc} \) is given by the reinforcement deformation relative to A-A shown shaded. In contrast, the force in the concrete in compression \( P_{cc} \) is given by the concrete deformation relative to C-C shown shaded. Similarly for \( P_{ct} \). The force in the tensile reinforcement \( P_{rt} \) depends on the half crack width \( \Delta \). This depends on the crack opening stiffness \( K \) that depends on the bond-slip stiffness \( k \) which can be obtained from a numerical analysis of the tension stiffening prism (Visintin et al 2013c) and for which closed form solutions are also available (Sturm et al 2016). For a given rotation \( \theta \), it is a question of varying \( d_{NA} \) until there is equilibrium.

![Figure 10 Level 4 Segmental analysis of beam at serviceability](image)

The material properties required for the analysis as listed in Figure 10(e) where: \( E_r \) is the reinforcement modulus; \( E_c \) the concrete modulus allowing for creep if necessary; \( \varepsilon_{sh} \) the shrinkage strain which can be assumed constant but the analysis can cope with non-linearity if necessary as shown; and the PI material bond-slip stiffness \( k \) which can be assumed to be constant at serviceability although it is very easy to allow for non-linearity if required. The segmental analysis in Figure 10 is a Level 4: Numerical Mechanical model which can be used in a one off analysis or in a parametric study to develop Level 5: Semi-Mechanical models for design.
Level 7 Model

First consider the case without shrinkage as in the segment in Figure 11(b). An Euler-Bernoulli deformation \( C - C \) is applied. The concrete in tension immediately below the neutral axis \( d_{NA} \) will be ignored as is common practice.

Above the neutral axis in Figure 11(a) there is full interaction (FI) that is there is no interface slip. Hence the resistance to deformation above the neutral axis can be determined by the commonly used transformed section as shown above the neutral axis in Figure 11(c) where the FI modular ratio is given by

\[
n_{FI} = \frac{E_r}{E_c}
\]

Below the neutral axis there is partial interaction so the full interaction modular ratio \( n_{FI} \) cannot be used. The force in the tension reinforcement \( P_{rt} \) in Figure 11(b) is equal to \( K\Delta \) where \( K \) is the crack opening stiffness and \( \Delta \) the extension as shown. From this PI mechanics can be derived the following PI modular ratio (Sturm et al 2016).

\[
n_{PI} = n_{FI} \sqrt{\frac{\beta k S_{cr}}{2}}
\]

in which \( \beta \) is the following prism axial rigidity factor

\[
\beta = \frac{S_{cr}}{2} \left( \frac{1}{E_r A_{rt}} + \frac{1}{E_c A_{ct}} \right)
\]

It can be seen in Eq 2 that the second factor on the right hand side allows for the beneficial effect of tension stiffening that is the increase in the full interaction modular ratio due to tension stiffening.

From the elementary analysis of transformed sections, the curvature at an applied moment \( M \) is given by

\[
\chi_{trans-3} = \frac{M}{E_c I_{trans-cr}}
\]

where the moment of inertia is

\[
I_{trans-3} = n_{PI} A_{rt} d (d - d_{NA}) - n_{FI} A_{rc} c_c (d_{NA} - c_c) - \frac{1}{b} d_{NA}^3
\]

in which

\[
d_{NA} = \sqrt{\left(\frac{n_{PI} A_{rt} + n_{FI} A_{rc}}{b}\right)^2 - \frac{2 n_{PI} A_{rt} d + n_{FI} A_{rc} c_c}{b} - \frac{n_{PI} A_{rt} + n_{FI} A_{rc}}{b}}
\]

The above PI transformed section analysis is mechanically exact and is, therefore, a Level 7 model. There are already closed form solutions for the crack spacing \( S_{cr} \) that depends on \( k \) (Sturm et al 2016) and, hence, the above procedure could be used directly in design should a suitable value of \( k \) be available for the new product. Alternatively, it could be used as a development tool to extract the material bond property \( k \) from member tests.

Level 6 Model

The transformed section analysis in Figure 11 can be applied when there is a single linear strain profile which does not occur when there are shrinkage strains such as in Figure 10. To determine a closed form solution for the effects of shrinkage, the compression zone in Figure 10 or 11 can be simplified to a full interaction prism as in Figure 12(a).

When the applied moment \( M \) in Figure 12(b) is zero, shrinkage causes the deformation \( D_{FI} \) that can be obtained from FI mechanics and the deformation \( D_{PI} \) that can be obtained from PI mechanics; this is shown as the Euler-
Bernoulli deformation $C_1-C_1$. From these deformations can be obtained the following curvature due to shrinkage alone (Visintin et al 2016)

$$\chi_{sh-1} = \frac{\varepsilon_{sh}}{d_r} \left[ \frac{E_{cc}A_{cc}}{E_{cc}A_{cc} + E_{rc}A_{rc}} - 1 + \frac{2}{\sqrt{\beta k}} \tanh \left( \frac{\sqrt{\beta k} S_{cr} / 2}{d_r} \right) \right]$$

in which $d_r$ is the distance between the tension and compression reinforcement. Now applying a moment which causes an increase in rotation $\theta_{sh-2}$ such that the deformation $D_{FI}$ now equals $\varepsilon_{sh}E_{rc}S_{cr}/2$ as shown in C2-C2 causes the following increase in curvature

$$\chi_{sh-2} = \frac{\varepsilon_{sh}}{d_r} \left[ \frac{2E_{rc}A_{rc} \tanh \left( \frac{\sqrt{\beta k} S_{cr} / 2}{d_r} \right)}{\sqrt{\beta k}E_{rc}A_{rc}S_{cr}} - \frac{1-E_{cc}A_{cc}}{E_{cc}A_{cc} + E_{rc}A_{rc}} \right]$$

which occurs when the applied moment $M$ is

$$M_{sh-2} = \varepsilon_{sh}E_{rc}A_{rc}d_r$$

which also depends on the shrinkage strain.

At the Euler-Bernoulli deformation $C_2-C_2$ in Figure 12(b), the strain in the compression reinforcement is simply the concrete shrinkage strain $\varepsilon_{sh}$ such that the force in the compression reinforcement is given by

$$P_{cc-sh} = \varepsilon_{sh}E_{rc}A_{rc}$$

which represents a constant residual force of whose affect is catered for by Eqs 7 and 8. As the effect of shrinkage has now been catered for, any further increase in rotation $\theta_{trans-3}$ in Figure 12(b) due to the component of the applied moment $M$-$M_{sh-2}$ is given directly by Eq 4 that is from a transformed section analysis that is independent of the shrinkage strain.

**Level 6 Model including $S_{cr}$ simplification**

A further simplification to the mechanics models is to assume that an approximation to the primary crack spacing is

$$S_p = \frac{2}{\sqrt{\beta k}}$$

Substituting Eq 11 into Eqs 2 gives the following simplified modular ratio

$$n_{FI-simp} = 1.31n_{FI}$$

which on substituting into Eq. 5 gives the following moment of inertia

$$I_{trans-cr-simp} = 1.31n_{FI}A_{rt}d(d - d_{NA}) - n_{FI}A_{rc}c_c(d_{NA} - c_c) - \frac{1}{6}\pi d_{NA}^3$$

Furthermore substituting Eq 11 into Eqs 7 and 8 gives

$$\chi_{sh-1-simp} = \frac{\varepsilon_{sh}}{d_r} \left[ \frac{E_{cc}A_{cc}}{E_{cc}A_{cc} + E_{rc}A_{rc}} - 0.238 \right]$$

$$\chi_{sh-2-simp} = \frac{0.762E_{rc}A_{rc}}{E_{rc}A_{rc}S_{cr}} - \frac{1-E_{cc}A_{cc}}{E_{cc}A_{cc} + E_{rc}A_{rc}} \frac{\varepsilon_{sh}}{d_r}$$
It can be seen that the curvatures from Eqs 13-15 are now in a form that are very easy to apply and can be used directly in design. In conclusion, the total curvature is given by

\[ \chi_{\text{total}} = \chi_{\text{sh-1}} + \chi_{\text{sh-2}} + \chi_{\text{trans-3}} \] (16)

where the first two terms depend on the shrinkage strain and the third component is independent of shrinkage and all the components depend on the bond-slip stiffness \( k \). These equations could be used directly in a one off design or used to develop design rules. Furthermore, as they separate the effects of bond stiffness, shrinkage and creep they will be very useful in research to study the time effect properties of concrete.

ADVANCED DESIGN

In the simplified approach above, the curvatures are independent of the bond-slip stiffness \( k \) leading to incredibly simple design rules. However the crack width \( w_{cr} \) is given by

\[ w_{cr\text{-simp}} = \frac{1}{\sqrt{2k}} [0.762\varepsilon_{sh} \left(1 + \frac{A_{rc}}{A_{ct}}\right) + \frac{(M - M_{sh-2})(d - d_{NA})}{k_{\text{trans-3}}}] \] (17)

where it can be seen that the crack width does depend on the bond-slip stiffness \( k \). This could be used by FRP reinforcement manufacturers to ensure that the bond is adequate.

Consider moment-redistribution in a continuous beam (Oehlers et al 2016b) in which moment is being redistributed from the hogging region to the sagging region and in which the moment redistribution factor \( K_{MR} \) is \( \Delta M_{\text{hog}}/M_{\text{hog}} \) and which is often given as a percentage in codes. Purely from mechanics, the moment redistribution factor is given by

\[ K_{MR} = \frac{1}{1 + \frac{\theta_{\text{max}}}{\theta_{\text{max}L}}} \] (18)

where \( \theta_{\text{max}} \) is the rotation capacity at a moment \( M_{\text{max}} \) that is close to the maximum moment capacity, \( L \) is the span and \( EI \) is the flexural rigidity of the beam between hinges. There are mechanics closed form solutions available for all of these parameters. This can be used to develop design rules for FRP reinforced members. Importantly the main parameter that controls \( \theta_{\text{max}} \) is not the ability of the reinforcement to yield but the softening branch of the concrete which could if required be enhanced with the addition of fibres. There is no reason to restrict moment redistribution in FRP reinforced section to any greater degree than for steel reinforced sections.

Finally consider IC debonding in Fig. 4. Level 7 models are now available to predict the occurrence of IC debonding and should it occur whether it is stable or catastrophic (Oehlers et al 2016a and 2016b). Hence the structural designer now has the tools to optimise the retrofitting technique. Furthermore should it be found through mechanics that catastrophic IC debonding cannot be avoided by for example changing the FRP plating system then mechanical analytical solutions are available for quantifying the effects of adding mechanical connectors to the adhesively bonded plated area to assist when IC debonding occurs. Another words the structural engineer now has the tools for an advanced FRP retrofitting design.

CONCLUSIONS

1. The main aim of this research is to provide structural engineers with Levels 4 to 7 models that incorporate all the relevant partial interaction local mechanisms that control the behaviour of RC members. This will eliminate the huge cost of member testing and also much of the substantial costs of the development of design rules.
2. Consultants can use these partial interaction models to design one-off FRP members and one off FRP retrofitting techniques for a specific need. Or they can be used in national standards to provide wide ranging mechanics design rules.
3. Manufacturers can use these models to develop new FRP products at considerable reductions in costs as the only testing required are the material properties. Once these properties are determined by tests they can be used in the PI models to determine the RC behaviour and consequently optimise the product.
4. Researchers will benefit from these models as they provide a deep understanding of the interaction between material properties such as concrete creep and shrinkage and bond-slip which will allow a better extraction of these material properties.
5. Much of this mechanics work has already been done. By substantially reducing costs and providing a mechanics in depth understanding, we believe that this approach will encourage, expand and speed up the use of FRP.

NOTATION

\( A_{cc} \) cross sectional area of concrete in FI compression prism
A_{ct}  
- cross-sectional area of concrete in tension stiffening prism

A_{cc}  
- cross-sectional area of compression reinforcement

A_{ct}  
- cross-sectional area of tension reinforcement

b  
- width of beam

c_{c}  
- distance of compression reinforcement to compression face

c_{t}  
- distance of tension reinforcement to tension face

D_{FI}  
- deformation of compression prism due to shrinkage alone

D_{PT}  
- deformation of tension stiffening prism due to shrinkage alone

d  
- effective depth; distance from compression face to tension reinforcement

d_{0A}  
- depth of neutral axis

EI  
- flexural rigidity of beam in regions between hinges

E_{c}  
- elastic modulus of concrete allowing for creep

E_{cc}  
- elastic modulus of concrete in compression prism

E_{ct}  
- elastic modulus of concrete in tension stiffening prism allowing for creep

E_{r}  
- elastic modulus of reinforcement

E_{rc}  
- elastic modulus of reinforcement in FI compression prism

E_{rt}  
- elastic modulus of tension reinforcement

FI  
- full interaction; no slip across interface

FRP  
- fibre reinforced polymer

h_{cr}  
- interface crack width

I_{trans-cr}  
- cracked moment of inertia from transformed section

IC  
- intermediate crack

K  
- crack opening stiffness; P/\Delta

K_{MR}  
- moment redistribution factor

k  
- initial bond-slip stiffness

L  
- span of beam

M  
- moment; applied moment

M_{max}  
- moment at \theta_{max} which is close to the maximum moment capacity

M_{sh-2}  
- applied moment required for \theta_{sh-2}

n_{FI}  
- FI modular ratio; E_{r}/E_{c}

n_{PI}  
- PI modular ratio

P_c  
- force in concrete in compression

P_{c-sh}  
- residual force in compression reinforcement to overcome \varepsilon_{sh}

P_{ct}  
- force in concrete in tension above the tension stiffening prism

P_{rc}  
- force in compression reinforcement

P_{rt}  
- force in tension reinforcement at crack

PI  
- partial interaction; slip across an interface

RC  
- reinforced concrete

S  
- interface slip

S_{cr}  
- crack spacing

\beta  
- axial rigidity constant

\chi  
- curvature; \theta/(S_{cr}/2)

\chi_{sh-1}  
- curvature due to shrinkage alone

\chi_{sh-2}  
- increase in curvature due to M that is affected by shrinkage

\chi_{total}  
- total curvature in a segment

\chi_{trans-3}  
- curvature from transformed sections

\chi_{trans-3}  
- increase in curvature due to M that is not affected by shrinkage

\Delta  
- slip of reinforcement relative to crack face; half crack width

\delta  
- bond slip

\varepsilon_{sh}  
- shrinkage strain

\theta  
- rotation; \chi S_{cr}/2

\theta_{max}  
- maximum rotation capacity beyond which moment reduces rapidly

\theta_{ba-1}  
- rotation due to shrinkage alone

\theta_{ba-2}  
- increase in rotation due to M that is affected by shrinkage

\theta_{trans-3}  
- increase in rotation due to M that is not affected by shrinkage

\sigma_{N}  
- interface confinement

\tau  
- interface shear; bond shear stress
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FIRE BEHAVIOUR OF FRP STRUCTURES

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ABSTRACT

Fibre-reinforced polymer (FRP) composites in general and pultruded glass-fibre-reinforced polymer (GFRP) profiles in particular are finding increasing applications in civil engineering, due to the several advantages they offer over conventional construction materials. However, for some applications, widespread use is being hindered due to concerns about their behaviour when subjected to elevated temperature and, especially, under fire exposure. These well-founded concerns are particularly acute for building applications, where relatively strict requirements need to be fulfilled in terms of fire reaction and fire resistance performance. With respect to bridges, fire is generally not a concern; yet, in some (special) cases, owners may also set fire behaviour requirements in tender specifications.

This paper presents a state-of-the-art review about the fire performance of pultruded GFRP structures, including the main research contributions from the author in the past 12 years. First, the effects of elevated temperatures on the thermo-physical and mechanical properties of the GFRP material are addressed. Subsequently, the fire reaction behaviour of the GFRP material (Figure 1) and the effects of using different types of fire protection on such behaviour are discussed. Next, the available fire resistance tests on pultruded GFRP elements, including slabs, beams (Figure 2) and columns, are reviewed and discussed, as well as the available modelling studies that aimed at predicting their thermal (Figure 3) and mechanical responses at elevated temperatures. Finally, international guidance for the fire design of pultruded FRP structures is summarized, and the most relevant research needs in this field are identified.

Figure 1 – Fire reaction test on a pultruded GFRP laminate.
Figure 2 – Different stages of a fire resistance test on a pultruded GFRP beam.
Figure 3 – Thermal (CFD) model of a GFRP profile exposed to fire.

KEYWORDS
FRP, pultruded profiles, elevated temperature properties, fire reaction, fire resistance, testing, modelling, design.
Bond Behaviour
DIRECT MEASUREMENT OF TRACTION-SEPARATION LAW OF CONCRETE/EPOXY INTERFACES UNDER MODE-I LOADING

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ABSTRACT

A proper traction-separation law based on the physical behaviors of epoxy/concrete interface is needed to simulate the debonding behavior of fiber reinforced polymers (FRP)-strengthened concrete structures. This study proposes a novel method using a wedge splitting specimen to directly measure the traction-separation law of the concrete/epoxy interface. This method requires a very simple testing set-up to produce stable crack propagation in the specimen. By adopting a rigid body moment assumption, the traction in the concrete/epoxy interface can be expressed as a function of the applied load and the crack opening displacement. In this way, the relationship between the traction and the separation of the concrete/epoxy interface can be directly measured through recording the crack opening displacement and the applied load during the test. Experimental results show that the traction-separation law of the concrete/epoxy concrete under mode-I loading can be approximated by a tri-linear law. This new method is also used to evaluate the long-term durability of concrete/epoxy interface subjected to moisture attack.

KEYWORDS

Debonding, environmental deterioration, fracture toughness, interface, traction-separation law.

INTRODUCTION

The traction-separation law of the concrete/epoxy under mode-II loading has been studied extensively. However, very few methods can be used to produce the traction-separation law of the concrete/epoxy interface under mode-I loading. To address this gap in the literature, this study presents a simple method to directly measure the traction-separation law of the concrete/epoxy interface under mode-I loading using a wedge splitting test (WST).

The long-term behaviors of the concrete/epoxy interface subjected to aggressive environments threats have become the focus of recent studies. Experimental studies show that exposure to moisture for a long time is the foremost factor that can degrade the interface. Substantial loss of fracture toughness of the interface induced by moisture was observed. In this study, the proposed new testing method will be used to evaluate the effect of water invasion on the concrete/epoxy interface. Organofunctional silane coupling agents (SiH4) have been proved to be useful to enhance the bond between an organic material such as epoxy and an inorganic material such as metal for long time. A few studies (Amidi and Wang 2016; Ye et al. 1998) also suggest that silane can be used to enhance the bonding between concrete and epoxy. Therefore, this role of silane will be examined by the proposed method.

MATERIALS AND METHOD

Wedge Splitting Test

The wedge splitting test is chosen because of its advantages over other popular fracture testing methods. WST requires very simple testing set up. The specimen is easy to fabricate and therefore, even can be done on-site or drilled from the structure. More importantly, the crack propagation in this specimen is stable, which eliminates the difficult task to control the crack growth in the other existing methods such as three-point bending test (Qiao and Xu 2004). The WST specimen consists of three parts: two identical concrete blocks, and a thin layer of cured epoxy bonding these two concrete blocks together, as shown in Figure 1. Since debonding is unlikely to occur
along the interface between the FRP and the adhesive layer, no FRP sheet was used in the specimen shown in Figure 1.

A commercially available silane coupling agent based on γ-glycidoxypropyltrimethoxysilane (γ-GPS) was used in this study to enhance the durability between the adhesive and the concrete. 1% silane solution (aqueous) was prepared. After hydrolysis for 1 hour in room temperature, the silane solution was continuously brushed onto the bonding surface of the concrete block for 10 minutes. The treated concrete blocks were then immediately dried in a preheated oven at 93 °C for one hour. Then the dried blocks were removed from the oven and cooled down to ambient temperature in the laboratory.

The testing set-up for the WST is shown in Figure 2a. The specimen is placed on a roller support mounted on a MTS machine. A steel wedge with two identical parallel plates is fixed at the top of the specimen to transfer the vertical force \( P_v \) applied by the MTS machine to two parallel roller supports at the top of the pre-crack region (Figure 2b). A splitting force \( P_s \) which is the horizontal component of the forces acting on the specimen \( P_N \) and friction force \( \mu P_N \) can be generated to drive two rollers apart (Figure 2b), leading to a mode-I fracture in the specimen. Digital Imagine Correlation (DIC) method is used to directly measure the crack opening of the specimen.

The testing set-up for the WST was first conditioned in a water tank at room temperature (23°C) for 4, 8 and 16 weeks before testing. In addition to the untreated specimens, specimens treated with silane coupling agent were also conditioned in water and then tested to evaluate the role of covalent bonds provided by silane in improving the resistance to water invasion of the concrete/epoxy interface. As a reference, a control group of specimens were conditioned in the ambient environment (with temperature (23°C) and relative humidity level of 50%). Four duplicated specimens were used for each test. The data reported in this study are average of these duplicated specimens.

**Testing Plan**

Inspired by the rigid double cantilever beam technique used for adhesive by Dastjerdi et al. (2013), a new direct measurement method of the traction-separation law of the concrete/epoxy interface using WST is proposed. Since concrete block is much stiffer than the adhesive layer, we can assume that the major deformation of the specimen is within the adhesive layer and the interface. Then the movement of the left and right concrete blocks of the specimen can be approximated as rotating about the symmetric axis like rigid bodies during testing, as shown in Figure 3. In such a case, we can assume that crack opening \( w(y) \) at position \( y \) (Figure 3) can be expressed by a linear function as

\[
w(y) = \frac{\delta}{h}y.
\]
where $\delta$ is the crack opening displacement (COD) at the crack mouth, and $h$ is the crack length. Dastjerdi et al. (2013) have confirmed through theoretical analysis that such a rigid body assumption is reasonable for short and stiff beams, which is the case of the WST specimen used here.

Since each half of the specimen can rotate about the vertical symmetric axis as a rigid body, its free-body diagram can be presented as shown in Figure 3. In this figure, the peel stress within the interface $\sigma(w(y))$, is a function of the separation of $w(y)$ of the interface. The equilibrium condition of the wedge in the vertical direction shown in Figure 2b gives

$$\eta P_N \cos \theta + P_N \sin \Phi = \frac{P_v}{2},$$

(2)

where $\eta$ is the friction constant between the roller and the wedge, $P_N$ is the reaction force from the roller to the wedge. Considering the equilibrium condition of the roller shown in Figure 2b in the horizontal direction, we can obtain the splitting force $P_{sp}$ applied by the wedge to the specimen as

$$P_{sp} = P_N \cos \Phi - \eta P_N \sin \Phi.$$  

(3)

Therefore,

$$P_{sp} = \frac{1 - \eta \tan \Phi}{2(\eta + \tan \Phi)} P_v,$$

(4)

where $\eta$ ( = 0.2) is the friction constant between the roller and the steel wedge; $\phi$ is a the angle of the steel wedge shown in Figure 2b.

**Figure 3 Free-body diagram of the half specimen**

Considering the moment equilibrium of the specimen about the right bottom corner of the half-specimen shown in Figure 3, we have

$$t \int_0^h \sigma(w(y))wdy = P_{sp}d_2 + \frac{1}{2} P_v d_1 + \frac{1}{2} mg e,$$

(5)

where $d_1$ and $d_2$ are the lever arms of force $P_v$ and $P_{sp}$ as shown in Figure 3, respectively; $mg$ is the self-weight of the concrete block; and $e$ is the distance of the center of the half specimen to the symmetric axis, as shown Figure 3.

Considering Eq. (1), Eq. (5) can be rewritten as

$$\int_0^\delta \sigma(w)wdw = \frac{1}{t} \left( \frac{h}{2} \right)^2 \left( P_{sp} d_2 + \frac{1}{2} P_v d_1 + \frac{1}{2} mg e \right).$$

(6)

Taking derivatives on both sides of Eq. (6) with respect to $\delta$ yields

$$\sigma(\delta) = \frac{2}{h^2 t} \left( P_{sp} d_2 + \frac{1}{2} P_v d_1 + \frac{1}{2} mg e \right) + \frac{\delta}{h^2 t} \left( \frac{dP_{sp}}{d\delta} \frac{d_2}{d\delta} + \frac{dP_v}{d\delta} \frac{d_1}{d\delta} \right).$$

(7)

Equation (7) directly gives the relationship between the separation $\delta$ and the traction (stress) $\sigma$ of the epoxy-concrete interface. It requires neither a pre-assumed shape for the traction-separation curve, nor any complicated inverse analysis. By measuring the COD and the applied load, the traction-separation law for epoxy-concrete interface under mode-I can be directly calculated using Eq. (7). The total fracture energy of the epoxy-concrete interface under mode-I loading can then be calculated from the traction- separation law as

$$G_I = \int_0^\delta \sigma(\delta) d\delta.$$  

(8)
TESTING RESULTS AND DISCUSSION

WST was conducted with a MTS machine under displacement control at the rate of 0.13 mm/min. A digital image correlation (DIC) system was used to take pictures of the specimens every 15 seconds. Images captured during the test were used to derive $\delta$ by simply reading the difference between displacements of two predetermined points at each half-specimen adjacent to the tip of the pre-crack. Figure 4 shows a typical result of the measured splitting force $P_{sp}$ versus COD $\delta$ for a WST specimen conditioned in water for 16 weeks. In order to extract the traction-separation law, the measured $P_{sp}$-COD curve was first smoothened to make it possible to calculate the derivative of $P_{sp}$ with respect to COD in Eq. (7), as shown by the red line in Figure 4. A typical result of such traction-separation law is shown in Figure 5 for a specimen conditioned in water for 8 weeks. It can be seen that the traction within the interface appears to increase with the separation initially. After reaching its maximum, the traction starts to drop with the separation, as shown in Figure 5. This traction-separation law can be approximated by a tri-linear traction-separation law shown in Figures 5 and 6.

![Figure 4](image1.png)

Figure 4 Splitting load versus COD measured at the crack mouth ($\delta$) for a specimen conditioned in water for 16 weeks.

![Figure 5](image2.png)

Figure 5 Measured traction-separation law of the epoxy-concrete interface under mode-I for a specimen conditioned in water for 8 weeks.

The resulted tri-linear traction-separation laws are shown in Figure 6. The control specimen exhibits much higher maximum tensile traction than the conditioned specimens. Silane coupling agent can improve the bond quality by forming a covalent bond of Si-O-Si between the epoxy and the concrete. A comparison at the beginning segment of the traction-separation law shows that conditioning in water can significantly reduce the initial stiffness of the bond, making it remarkably softer in comparison with the control specimens, as shown in Figure 6c. This is exacerbated when longer conditioning duration is used. Silane coupling agent treatment can slightly improve the stiffness of the interface after short-term conditioning in water, as shown in Figure 6b.
Figure 6 Tri-linear traction-separation laws measured for concrete/epoxy interface after conditioning in water: a) effect of conditioning in water on the traction-separation law; b) effect of silane on the traction-separation law of the interface after conditioning in water; c) effect of water conditioning on the stiffness of the concrete/epoxy interface

Using Eq. (8), the mode-I fracture toughness for each group can be obtained and presented in Figure 7. After conditioning in water for 4 weeks, the mode-I fracture toughness of all untreated specimens have been decreased remarkably compared to the control ones.
Figure 7 Effect of duration of water conditioning on the fracture energy of the concrete/epoxy interface with and without silane coupling agent

CONCLUSIONS

In this study, a novel testing method using WST specimen is proposed to directly measure the traction-separation law of the concrete/epoxy interface under mode-I loading. This new method is then used to evaluate the long-term durability of the concrete/epoxy interface subjected to moisture attack. Experimental study suggests that the traction-separation law of the concrete/epoxy interface under mode-I loading can be conveniently obtained using this new method. The traction-separation law of the concrete/epoxy interface under mode-I loading can be approximated by tri-linear laws. Conditioning the specimens in water can significantly reduce the tensile strength and the fracture toughness of the interface. By applying saline coupling agent, the residual strength and fracture toughness of the concrete/epoxy interface after conditioning in water are higher than that of the specimens without silane treatment, suggesting that the silane coupling agent is effective to improve the long-term durability of the concrete/epoxy interface subjected to moisture attack.

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ABSTRACT

The on-site evaluation of the bond quality between externally bonded Fiber-Reinforced Polymer (FRP) composites and concrete structures is crucial to assess the performance and predict the durability of the reinforcement system. In this study, it is proposed to determine the bond properties of the adhesive layer by using a nondestructive testing (NDT) method derived from the standard pull-off test. This method consists in analyzing the linear load vs displacement behavior of the adhesive joint subjected to a mixed-mode loading, in order to determine a bond stiffness which could be related to the bond quality. This paper gives some insight into the proposed test method applied to concrete slabs reinforced with Carbon-FRP (CFRP) plates bonded with three different epoxy adhesives. The experimental implementation and the first results are shown and discussed.

KEYWORDS

FRP composites, concrete, bond quality, non-destructive evaluation, pull-off test.

INTRODUCTION

Over the last 30 years, structural reinforcement and retrofitting with externally bonded composite materials has proven to be an efficient and cost-effective solution to increase both the safety and lifespan of civil engineering structures (ISIS Canada Research Network, 2007). In the composite – adhesive – concrete assembly, the stress is transferred from the structure to the composite material through the shear deformation of the adhesive layer. Therefore, for the sake of performance and durability, the level of adhesion between the FRP material and concrete substrate must be of good quality (Ferrier et al, 2010).

A common way to assess the bond quality in the field is to use the standardized direct tension pull-off test (ASTM D7522 / D7522M, 2015; EN 1542, 1999), pictured on Figure 1.a. It consists in bonding a steel fixture called dolly at the FRP surface and drilling a partial core around it. The dolly is then loaded perpendicularly to the FRP surface, until failure occurs at the so-called pull-off strength. A sound assembly shows a cohesive failure in the concrete substrate with a pull-off strength higher than a minimal value (usually 1.5 or 2 MPa, depending on the guidelines). However this destructive method has many drawbacks, as there is a large scattering on test results and the pull-off strength is highly influenced by the concrete properties and the geometrical characteristics of the partial core.

The present study intends to develop an alternative nondestructive (ND) test method which is able to evaluate the level of adhesion, and is suitable for use in the field. The proposed method is derived from the standard pull-off test, but an additional equipment provides measurements of the dolly displacements with a high accuracy, and the test is only carried out in the linear elastic domain. It is thus possible to plot the load vs displacement curve, whose slope relates to the assembly stiffness. The scope of this first paper is to show the feasibility of this method for providing quantitative and discriminating data on the stiffness of the concrete/CFRP bonded assembly that can be further interpreted in terms of bond quality.

DESCRIPTION OF THE NONDESTRUCTIVE TEST METHOD

This feasibility study is restricted to the case of composite plate systems for simplicity reasons. Further researches should be conducted on other systems like fiber sheets bonded to the structure by wet lay-up. The proposed ND method is depicted on Figure 1.b. As mentioned before, it is based on the standard pull-off test, but there are three main differences: (1) no core is drilled around the dolly, (2) a displacement measurement is performed in addition
to the load measurement, and (3) the test is carried out in the linear elastic domain. From both measurements, a load vs displacement curve can be plotted, whose slope will be called the bond or assembly stiffness.

The displacement should be measured at a well-chosen location of the concrete/FRP bonded assembly, so that it provides relevant information on the bond behavior. A differential displacement is also an interesting way to focus on the most interesting contributions. In the present study, a differential displacement between the dolly and the concrete surface is considered (Figures 1.b and 2.a). In addition to the E-modulus of the adhesive, several parameters influencing this displacement and the bond stiffness have been identified, namely the thickness of the adhesive, the elastic properties of the concrete substrate, the E-modulus and the thickness of the bonding agent between the dolly and the composite plate. This issue will be addressed in the developments. Differently, the deformation of the steel dolly is considered negligible and the contribution of the pultruded composite plate is supposed well-known. It should be pointed out that micron-level displacements are expected.

As no core is drilled around the dolly, the adhesive joint is not subjected to pure tensile stress but to a mixed-loading mode similar to a peel test. This type of loading is actually experienced by the FRP systems in flexural strengthening applications (Buyukozturk et al., 2004), and by FRP systems bonded to curved substrates like arches or cooling towers (De Lorenzis and Zavarise, 2009). This mixed-loading mode is thus relevant for assessing the bond quality.

In order to investigate the feasibility of the ND pull-off test, an experimental campaign was conducted, which is described in the following sections.

EXPERIMENTAL PROGRAM

The aim of this experimental campaign is to check whether the ND pull-off test is able to distinguish between adhesive materials showing various Young’s moduli and simulating different levels of adhesion.

Experimental setup

In order to have precise control of the experimental setup at the metrological level, it was decided to carry out the tests on laboratory equipment instead of using a commercial pull-off tester. The ND pull-off test is conducted on a universal testing machine equipped with a load cell of capacity 50 kN. In order to firmly hold the concrete specimens, a clamping device, presented Figure 2.b, was designed using a commercial computed aided design (CAD) software, and then machined in a precision manufacture. This device consists of two overlapping steel plates and vises and allows two translational degrees of freedom for more flexibility in specimens’ geometry. A finite elements model of the apparatus was computed to understand its behavior during the loading phase and estimate, for all bolts, the torques needed to ensure that the different parts remain in contact, so that undesirable motions are minimized. The whole device was consequently assembled and installed on the universal testing machine using a torque wrench and following a specific procedure aiming to insure horizontality as much as possible.

As for the standard pull-off test, the load is applied to the dolly through a ball-joint fixture. For the present experimental campaign, the ball-joint system (a spherical headed bolt and a mechanical coupling) was supplied as spare parts of a commercial pull-off tester. Special care is given to the centering of the spherical headed bolt into the coupling, in order to minimize possible flexion loading.

Special steel loading fixtures were produced, whose geometry was based on standard dollies. On top of a 50 mm diameter cylinder was added a 110 mm diameter and 5 mm thick plateau, with total height of 25 mm (Figure 2.a). This plateau allows performing displacement measurements, which would not have been possible otherwise,
because of the bulk of the ball-joint coupling. Dollies are glued to the specimens using a contact cyanacrylate adhesive, which is rubber toughened and exhibits increased flexibility and peel strength. Its low viscosity ensures a contact bond, which minimizes the tensile deformation of the bond line and its contribution to the measured displacement, which was earlier identified as a parameter influencing the bond stiffness. The fixture time for a steel – CFRP interface is about 2 – 3 minutes, which gives enough time for positioning the dolly properly.

The displacement measurement between the dolly and the concrete surface is performed using high accuracy optical digital sensors, which provides consistent submicron measurement over a 12 mm displacement range. Two sensors are installed on each side of the system’s plane of symmetry, in order to get average values based on two displacement measurements (see Figure 2.a). The two sensors were calibrated prior the tests and presented expanded uncertainties of 0.9 µm and 0.6 µm respectively (coverage factor of 2). They were attached to the sides of the dolly’s plateau by means steel rings glued with a 2-component fast curing methacrylate adhesive.

Load measurement is provided by the load cell of the universal testing machine. However, it was necessary to retrieve the load data with the acquisition system of the displacement sensors in order to synchronize both measurements. This was carried out by means of a specific input / output board and an analog-to-digital converter. This load acquisition system was also calibrated prior to the tests and present an expanded uncertainty of 120 N. All experiments are performed at a controlled room temperature of 21°C ± 2°C.

Specimens

Different levels of adhesion were simulated using three commercially available epoxy adhesives, with Young’s moduli of 11,200, 4,950 and 965 MPa respectively. It is worth noting that Adhesives 1 and 2 are commercial systems intended for bonding CFRP plates on concrete structures, while Adhesive 3 is a low modulus system that simulates an environmentally aged epoxy adhesive, as a significant decrease in the elastic properties is usually observed after accelerated ageing in humid environments, due to a plasticization of the polymer network (Benzarti et al, 2011). The properties of the unidirectional CFRP laminate are listed in Table 1.

Table 1 Typical properties of the CFRP plates used in the present study

<table>
<thead>
<tr>
<th>Description</th>
<th>Thickness</th>
<th>Width</th>
<th>Fiber content</th>
<th>Modulus</th>
<th>Tensile strength</th>
<th>Elongation at break</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pultruded unidirectional (orientation 0°) CFRP plate</td>
<td>1.2 mm</td>
<td>80 mm</td>
<td>&gt; 68%</td>
<td>165,000 MPa</td>
<td>2,900 MPa</td>
<td>&gt; 1.80 %</td>
</tr>
</tbody>
</table>

The experimental program consists of eighteen concrete slabs strengthened with CFRP-laminates bonded using the three different adhesives (six specimens for each adhesive). Among each series of six specimens, three specimens (labelled “D” for destructive) were tested until failure. A limit load, lower than the recorded ultimate failure loads was then chosen. The load applied to the three others specimens (labelled “ND” for nondestructive) did not exceed this limit load, insuring the nondestructive nature of the tests. It is to note that displacement data acquired for four out of the eighteen specimens were not exploitable. The overall experimental program is summarized in Table 2.
Table 2 Experimental program

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Adhesive 1</th>
<th>Adhesive 2</th>
<th>Adhesive 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>D</td>
<td>ND</td>
<td>D</td>
<td>ND</td>
</tr>
<tr>
<td>1.1</td>
<td>×</td>
<td>×</td>
<td>×</td>
</tr>
<tr>
<td>1.2</td>
<td>×</td>
<td>×</td>
<td>×</td>
</tr>
<tr>
<td>1.3</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>1.4</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>1.5</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>1.6</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
</tbody>
</table>

Displacement data

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Adhesive 1</th>
<th>Adhesive 2</th>
<th>Adhesive 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>D</td>
<td>ND</td>
<td>D</td>
<td>ND</td>
</tr>
<tr>
<td>1.1</td>
<td>×</td>
<td>×</td>
<td>×</td>
</tr>
<tr>
<td>1.2</td>
<td>×</td>
<td>×</td>
<td>×</td>
</tr>
<tr>
<td>1.3</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>1.4</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>1.5</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>1.6</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
</tbody>
</table>

300 × 300 × 70 mm³ concrete slabs of type MC 0.45 were casted according to (EN 1766, 2000). The maximum aggregate size is 10 mm. Three concrete batches were mixed for a total of thirty-six blocks (the rest of the blocks will be used in a durability study). The 28 days compressive strength of concrete was evaluated on cubic specimens with 150 mm long edges, giving an average value of 63.5 MPa, which corresponds to a C45/55 strength class and an indicative compressive modulus of 36 GPa (EN 1992-1-1:2004). Due to the bad condition of the concrete surface, an extensive grinding treatment was achieved, until the surface was completely planar. The different steps of the specimen preparation are presented in Figure 3. For each specimen, a 300 mm long strip of composite plate is bonded at the center of the concrete slab. However, the edges of the concrete blocks were covered with a rubber tape of width 2 cm, in order to prevent edge effects. The total bonded length for the CFRP plate is therefore 260 mm.

![Grinded concrete surface](image1)
![Composite plate cut into pieces](image2)
![Application of the adhesive](image3)
![Steel mass pressing the bond line](image4)
![Final aspect of a specimen](image5)
![Set of specimens used for the present study](image6)
![Measurement of the adhesive thickness](image7)

Figure 3 Specimen preparation

As mentioned earlier, the thickness of the adhesive layer is expected to influence the overall bond stiffness. Special care was given to control the thickness as much as possible during specimen preparation. A steel mass with legs 3 mm high was used to press the composite plate and the adhesive layer in order to obtain a desired thickness of the adhesive joint. The final thickness in the central area was also measured with a caliper prior to the bonding of the dolly, as shown in Figure 3.g. Thickness values of the adhesive layer are given in Table 3 for specimen on which the displacement measurement could be performed. A value is the average of 9 point-like measurements in the central area of the composite plate. The associated uncertainty is calculated by summing the square experimental error (valued at 0.1 mm) and the standard deviation of the different measurements.

Table 3 Measured adhesive layer’s thickness with associated uncertainty

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Adhesive 1</th>
<th>Adhesive 2</th>
<th>Adhesive 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.4 1.5 1.6</td>
<td>2.1 2.2 2.3</td>
<td>2.4 2.5 2.6</td>
</tr>
<tr>
<td>Thickness (mm)</td>
<td>1.5 1.6 1.9</td>
<td>1.7 1.5 1.8</td>
<td>1.7 1.9 1.5</td>
</tr>
<tr>
<td>Uncertainty (mm)</td>
<td>0.1 0.2 0.1</td>
<td>0.1 0.2 0.1</td>
<td>0.1 0.1 0.1</td>
</tr>
</tbody>
</table>

Loading procedure

A specific loading procedure was followed during the tests. The loading is displacement-controlled, the crosshead of the testing machine moving at a speed of 0.3 mm/min. A cyclic protocol is applied, with a baseline at 200 N and the maximal load increasing every three cycles. No limit load was set for the “D” specimens, unlike “ND” specimens.
RESULTS AND DISCUSSION

Determination of the limit load

The failure loads for the nine “D” specimens tested until failure are given in Table 4. It should be noted that all specimens exhibited a brittle fracture, except for specimens 2.1 and 3.3. For these specimens, a loss in the slope of the load vs displacement curve, i.e. in the bond stiffness, could be observed. The failure load is defined as the load corresponding to the first brittle event associated with damage or to a loss in the bond stiffness. The nature of this event (type of damage) is also reported.

Table 4 Failure load and associated event for the “D” specimens, and limit load chosen for the different adhesives.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Adhesive 1</th>
<th>Adhesive 2</th>
<th>Adhesive 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.1</td>
<td>1.2</td>
<td>1.3</td>
</tr>
<tr>
<td>Failure load</td>
<td>14.0 kN</td>
<td>14.1 kN</td>
<td>-*</td>
</tr>
<tr>
<td>Event at failure</td>
<td>Debonding of the dolly</td>
<td>Debonding of the dolly</td>
<td>Loss in the bond stiffness + dull noise at 10 kN</td>
</tr>
<tr>
<td>Chosen limit load</td>
<td>10 kN</td>
<td>8 kN</td>
<td>8 kN</td>
</tr>
</tbody>
</table>

* The failure load of specimen 1.3 could not be recorded since the specimen was accidently damaged during installation.

As regard failures accompanied with a dull noise, a small bulge of the composite plate could also be observed in the area surrounding the dolly. This dull noise is due to the tensile failure of the concrete substrate underneath the dolly, which could be verified after the composite plate was completely pulled off.

Taking the previous results into account, the following limit loads were chosen for the “ND” specimens (also reported in Table 4): 10 kN for Adhesive 1, 8 kN for Adhesive 2 and 8 kN for Adhesive 3.

Load vs displacement curve and bond stiffness

The load vs displacement curves of all fourteen exploitable specimens (see Table 2) are presented in Figure 4. Only the last load cycle is displayed. In the present results, the average displacement of the two sensors is used.

![Figure 4 Load vs displacement behavior acquired during the ND pull-off tests.](image)

Values of the bond stiffness, corresponding to the slope of the load vs displacement curves, are also calculated through linear regression. Only the data until the limit load associated with each adhesive are considered, even for “D” specimens which provide further data. Calculated values of the bond stiffness are given in Table 5, together with a first estimation of the expanded uncertainty (coverage factor of 2). It is evaluated from the uncertainty on the displacement measurement using linear least-square formulae. The uncertainty on the load measurement is
neglected as it is relatively small compared to the uncertainty on the displacement. Repeatability and reproducibility uncertainties will also be taken into account in the future.

Table 5 Values of the Bond stiffness calculated by linear regression of the load vs displacement curves.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Adhesive 1</th>
<th>Adhesive 2</th>
<th>Adhesive 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bond stiffness ( \times 10^9 ) N/m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.4</td>
<td>1.5</td>
<td>1.6</td>
<td>2.1</td>
</tr>
<tr>
<td></td>
<td>1.96</td>
<td>1.95</td>
<td>1.95</td>
</tr>
<tr>
<td>Expanded uncertainty ( \times 10^9 ) N/m</td>
<td>0.03</td>
<td>0.02</td>
<td>0.03</td>
</tr>
</tbody>
</table>

From the previous results, the following observations can be made:
- It is possible to distinguish specimens according to the type of adhesive used for bonding CFRP plates, although there is an ambiguity for specimens 2.4 and 2.5 one the one hand, specimens 3.3 and 3.5 on the other hand, which show close bond stiffness values. Thus, the ND pull-off test can discriminate several levels of adhesion.
- Different slopes, i.e. different values of bond stiffness, are found for specimens produced with one given adhesive. However, this is unlikely due to statistical dispersion as several specimens can have very close bond stiffness values (specimens 2.1, 2.2 and 2.6 for instance).
- The adhesive thickness seems not to influence significantly the bond stiffness. For instance, specimens 1.4, 1.5 and 1.6 have thicknesses of the adhesive layer equal to 1.5 ± 0.1 mm, 1.6 ± 0.2 mm and 1.9 ± 0.1 mm respectively, but they exhibit very close values of the bond stiffness.
- Damaging of specimen 2.4 started just below 8 kN, which could be detected by a loss in the bond stiffness. However, voids in the adhesive layer located near the dolly area were detected by infrared thermography prior to the test. It is believed that the early failure of this specimen is due to the voids. Only the data collected before this loss of bond stiffness (under 6 kN) are presented in Figure 4 and are used to calculate the slope.

It is believed that differences in the apparent bond stiffness observed for a given type of epoxy adhesive are not related to statistical dispersion, but may result from variations in other influential parameters of the bonded assembly, and most likely the properties of the concrete substrate. As a reminder, three batches of concrete were used to cast all the slabs, which is consistent with the maximum of three different slopes observed in the case of specimens prepared with Adhesive 2. To confirm this hypothesis, further research will consist in conducting the ND pull-off test on the bare concrete surface in order to measure a reference stiffness for each batch of concrete.

**CONCLUSION AND PERSPECTIVES**

In this paper, a nondestructive test is proposed for assessing the bond quality in a concrete-to-FRP bonded assembly. It is based on the popular standard pull-off test, but shows three main differences: (1) no partial core is drilled around the dolly, (2) a displacement measurement is performed in addition to the load measurement, and (3) the test is carried out in the linear elastic domain. The bond quality criterion depends on the bond stiffness deduced of the load – displacement curve. The first results presented in this paper suggest the ability of the ND pull-off test to distinguish between different levels of adhesion. These are experimentally simulated by using different epoxy resin systems showing distinct Young’s moduli. Nonetheless, different values of bond stiffness are found for a given adhesive, and it is believed that a change in the concrete’s properties is responsible for this difference. It is therefore suggested to complete our experimental methodology by conducting the same ND pull-off test on the bare concrete surface, in order to measure a reference stiffness related to the concrete’s behavior. A finite element modeling will also be completed to help analyzing the behavior of the system and the test results.

This test method could be a promising tool to assess the bond quality of composite strengthening systems in the field. It could be carried out using a commercial pull-off tester, provided it is equipped with a sufficiently precise load cell and a displacement measurement system, and a synchronization is ensured between both measurements.

**REFERENCES**


DEFECT CRITICALITY IN FRP/CONCRETE BOND JOINT – A FINITE ELEMENT STUDY

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ABSTRACT
Defects, such as debonded region at FRP/concrete interface, can occur over time in fiber-reinforced polymer (FRP) strengthening system in concrete structures. Limited numbers of researches have showed that the overall performance of reinforced concrete members strengthened with FRP can deteriorate due to the presence of defects. This can result in reduction of load capacity, stiffness, and durability, potentially causing premature failure, if not treated properly. This study investigated the effects of defect (or defect criticality) at bond level, using finite element analysis of specimens used in single-lap shear test. The defect was in the form of debonded region at FRP/concrete interface. The parameters included bond width and bond length, size and location of defect, and the compressive strength of concrete ($f'_c$). The results have showed that, in most cases, the distribution of interfacial shear stress and bond strength were affected by the presence of defects. Bond strength reduction was more pronounced in FRP/concrete bond joint with bond length smaller than the effective value or with higher FRP stiffness.

KEYWORDS
Defect criticality, bond strength, bond-slip relationship, FRP/concrete interface.

INTRODUCTION
Fiber reinforced polymer (FRP) composites have been increasingly used to enhance load capacity and improve serviceability of existing reinforced concrete (RC) members and structures in the last two decades. This is due to their advantages over other conventional strengthening materials, such as higher strength-to-weight ratio, corrosion resistance, and ease of installation. For flexural strengthening of RC beams, FRP material is externally bonded to the tension face of the RC member using adhesive, or embedded in concrete in case of near-surface mounted FRP rods. In spite of ample research on short-term mechanical behavior, this strengthening technique still cannot realize its full potential due to limited knowledge on its long-term durability and criticality of potential defect. Due to its composite nature, the effectiveness of the FRP strengthening system for flexure and shear depends on the performance of the FRP/concrete interface. However, defect in the form of debonded regions can exist at the FRP/concrete interface in flexurally and shear strengthened members as a result of poor construction, or subsequent physical damage and environmental degradation during its intended service life after rehabilitation. Limited numbers of studies have showed that the presence of defects can have detrimental effects on FRP-concrete systems (Seim et al. 2001, Karbhari and Navada 2008, Kalayci et al. 2009, Guo et al. 2012). This paper is aimed at gaining more understanding on the effect of defect on the FRP/concrete system at bond joint level using finite element analysis (FEA). The parameters included bond width and bond length, size and location of defect, and the compressive strength of concrete ($f'_c$).

METHODOLOGY

Bond-Slip Relationship for FRP/Concrete Interface
In simulating debonding failure of FRP/concrete bond joint in FEA, the relationship between shear stress ($\tau$) and slip ($\delta$) in the FRP/Concrete interface can be described by a bond-slip model. Several forms of bond-slip model have been proposed in the past 10 years based on analysis of experimental results from fracture tests (Nakaba et al. 2001, Lu et al. 2005). The simplest form is a bi-linear bond-slip curve consisting of linear ascending and descending branches, which can be described by three parameters: maximum interfacial shear stress ($\tau_m$), slip at maximum interfacial shear stress ($\delta_0$), and maximum slip ($\delta_f$) (Figure 1a). The linear ascending branch describes...
the stress-slip relationship in elastic regime when \(0 \leq s \leq s_0\), which is expressed by Eq. 1. In this regime, the bond joint will behave elastically until the interfacial shear stress and slip reach \(\tau_m\) and \(s_0\), respectively. The linear descending branch or softening branch describes the behavior of damaged interface (due to micro-cracking and crazing in the adhesive) after shear slip exceeds \(s_0\). As the value of slip increases beyond \(s_0\), the interfacial shear stress decreases linearly according to Eq. 2. Debonding of the FRP/concrete bond joint takes place when \(s > s_f\), after which the interfacial stress becomes zero, signifying the inability of the interface in the debonded region to transfer load between FRP and concrete.

\[
\tau = \tau_m \left( \frac{s}{s_0} \right) \quad \text{for} \quad 0 \leq s \leq s_0
\]

(1)

\[
\tau = \tau_m - \frac{\tau_m (s - s_0)}{s_f - s_0} \quad \text{for} \quad s_0 \leq s \leq s_f
\]

(2)

Figure 1 A typical bi-linear bond-slip relationship (a) and single-lap shear specimen (b)

Ko et al. (2014) have recently developed a bond-slip relationship for practical design purpose by calibrating the afore-mentioned model parameters using database of various single-lap shear and double-lap shear test results. According to Ko et al. (2014), the parameters for the bi-linear bond-slip model may be calculated from Eqs 3 through 5.

\[
\tau_m = 0.165f'_c
\]

(3)

\[
s_0 = -0.001f'_c + 0.122
\]

(4)

\[
s_f = -0.002f'_c + 0.302
\]

(5)

where \(f'_c\) is the compressive strength of concrete (MPa). The maximum interfacial shear stress and slip parameters are given in MPa and mm units. This study used the bond-slip relationship proposed by Ko et al. (2014) to simulate the debonding behavior of the CFRP/concrete interface.

Figure 2 FE models without defect (a) and with defect of size 10 mm at midpoint in bond (b)

Finite Element Model of Single-lap Shear Test Specimen

In this study, 2-dimensional FE models of FRP/concrete bond joint were constructed based on the specimens in single-lap shear test conducted by Yao et al. (2005). Figure 1b shows the schematic representation of single-lap shear test, and Figure 2 shows the FE models with and without defect. The models consisted of 4-node plane-stress elements (CPS4) for concrete block and FRP plate, and 4-node cohesive elements for interface layer (COH2D4). Note that various tests have shown that failure in FRP/concrete bond joint is mainly in the form of separation at concrete/adhesive interface with thin concrete layer remaining on adhesive surface. Therefore, the interface layer in the FE models with bond behavior governed by the specified bond-slip relationship will represent this failure zone in real specimens. The 4-node cohesive elements used in this study are capable of describing both normal (mode I) and shear (mode II) deformations as shown in Figure 3. The relationship between normal deformation (\(n\)) and normal stress (\(\sigma\)) can be specified by bi-linear relationship similar to the bond-slip relationship for shear behavior in Figure 1a (Eqs 1 and 2). However, since the interface in single-lap shear test mainly deforms
in shear, debonding by mode I fracture was prevented in the FE models by specifying a value of maximum normal stress $\sigma_m$ significantly higher than the maximum interfacial shear strength $\tau_m$ (e.g. 2 orders of magnitude higher).

Both concrete and FRP used linear elastic material behavior, with compressive strengths of concrete equal to 23 and 27.1 MPa, and Young’s modulus of FRP equal to 256 GPa. The Young’s moduli of concrete ($E_c$) at the two strength levels were calculated according to the ACI Guideline (2008) to be 22.68 and 24.62 GPa, respectively. According to Eqs 3 and 4, the initial stiffness of the interface layer are 38.33 and 47.12 MPa/mm, and the maximum interfacial shear stress ($\tau_m$) are 3.80 and 4.47 MPa, for the two concrete strength levels. Figure 4 compares bond strengths obtained from FEA (without defect) and from the experiment reported in Yao et al. (2005). It suggests that the proposed FE model could adequately simulate the strength and behavior of FRP/concrete interface in single-lap shear test.

In order to investigate the effect of defect, debonded region was inserted into the interface by omitting cohesive elements in the corresponding region for a specified length. Without cohesive elements, there is no shear transfer between FRP and concrete in defect area. Figure 5 shows the characteristics of defect, including length ($L_d$) and
location along the bond line. The size of concrete block, free concrete height ($h_c$), bond length ($L_d$), and FRP thickness were based on the single-lap shear specimens without defect in the study by Yao et al. (2005). Table 1 shows model groups with corresponding geometries. $L_a$ and $a$ in the model codes are defect size and location, respectively, with $L_a = 0$ and $a = 0$ meaning no defect in the interface. Locations of defect are denoted by $LE$ for defect located close to the loaded end, $L/4$ for defect located at quarter length closer to the loaded end, $L_f$ for defect located at the center of bond joint, $3L/4$ for defect located at quarter length closer to the free end, and $FE$ for defect located close to the free end. The sizes of defect $L_a$ in this study were 10, 20, 40 mm.

<table>
<thead>
<tr>
<th>Model</th>
<th>$f_c$ (MPa)</th>
<th>$b_i$ (mm)</th>
<th>$b_f$ (mm)</th>
<th>$L_d$ (mm)</th>
<th>$h_c$ (mm)</th>
<th>Model</th>
<th>$f_c$ (MPa)</th>
<th>$b_c$ (mm)</th>
<th>$b_f$ (mm)</th>
<th>$L_f$ (mm)</th>
<th>$h_i$ (mm)</th>
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<tbody>
<tr>
<td>A- $L_d$-a</td>
<td>23</td>
<td>150</td>
<td>25</td>
<td>75</td>
<td>5</td>
<td>F- $L_d$-a</td>
<td>27.1</td>
<td>150</td>
<td>25</td>
<td>100</td>
<td>120</td>
</tr>
<tr>
<td>B- $L_d$-a</td>
<td>23</td>
<td>150</td>
<td>25</td>
<td>95</td>
<td>5</td>
<td>G- $L_d$-a</td>
<td>27.1</td>
<td>150</td>
<td>50</td>
<td>100</td>
<td>120</td>
</tr>
<tr>
<td>C- $L_d$-a</td>
<td>23</td>
<td>150</td>
<td>25</td>
<td>115</td>
<td>5</td>
<td>H- $L_d$-a</td>
<td>27.1</td>
<td>150</td>
<td>75</td>
<td>100</td>
<td>120</td>
</tr>
<tr>
<td>D- $L_d$-a</td>
<td>23</td>
<td>150</td>
<td>25</td>
<td>145</td>
<td>5</td>
<td>I- $L_d$-a</td>
<td>27.1</td>
<td>150</td>
<td>100</td>
<td>100</td>
<td>120</td>
</tr>
<tr>
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<td>150</td>
<td>25</td>
<td>190</td>
<td>5</td>
<td>J- $L_d$-a</td>
<td>27.1</td>
<td>100</td>
<td>23.5</td>
<td>100</td>
<td>120</td>
</tr>
</tbody>
</table>

**RESULTS AND DISCUSSION**

**Behavior of FRP/concrete Bond Joint with and without Defect**

In both models with and without defect, the interfacial shear stress ($\tau$) distribution can be categorized into three regimes – Elastic, Elastic-softening, and Elastic-Softening-Debonding. These three behaviors depend on the magnitude of the applied load. At low load level, behavior of the interface is Elastic. Elastic-softening behavior starts just after the applied load exceeds the softening load ($P_s$). As the load increases beyond $P_s$, local debonding will take place (slip $> s_0$) and the behavior becomes Elastic-Softening-Debonding. The ultimate load or bond strength $P_c$ corresponds to when shear stress in the entire interface becomes zero. Figure 6 shows samples of interfacial shear stress distributions from models E-0-0 and E-10-C. Note that by examining the stress distribution in perfect models of various bond lengths, the effective bond length for this particular FRP/concrete system was approximately 190 mm. When comparing stress distribution between bond joints with and without defect, it has been observed that defect can cause slightly increase in stress magnitude, depending on size and location. Figure 7 shows the effect of defect of various locations and sizes on the shear stress distribution for the models based on D-0-0. The increase in shear stress is more pronounced in the vicinity of defect (see Figure 7). This can be attributed to the reduction in the true bond length. Larger defects ($L_d = 40$ mm) resulted in generally higher increase in overall shear stress. In addition, defects at locations of high stress normally in perfect bond (e.g. $L/4$ and $C$) resulted in higher increase in shear stress. This may be because there is high shear transfer between FRP and concrete in these locations. Therefore, removing a portion of the interface in this zone will result in higher shear transfer in the proximity.

![](image)

Figure 6 Sample of interfacial shear stress distribution under varying applied load: (a) Model without defect (Model E-0-0 with with $f'_c = 23$ MPa, $b_i = 150$ mm, $b_f = 25$ mm, $L_d = 190$ mm, and $h_c = 5$ mm) and (b) Model with defect (Model E-10-C with $L_d = 10$ mm and defect at the center of bond line)
Figure 7 Sample of interfacial shear stress distribution as affected by: (a) Defect sizes (Models D-0-0, D-10-C, D-20-C, and D-40-C) and (b) Defect locations (Models D-0-0, D-10-LE, D-10- Lf/4, D-10-C, D-10-3Lf/4 and D-10-FE)

**Effect of Defect on Bond Strength**

Figure 8 shows the bond strength as affected by defect size and location in the FRP/concrete bond joint with varying $L_f$. Larger defect resulted in higher reduction in bond strength, which is consistent with the increase in interfacial shear stress. For the defect of the same size, defects at the center and at $L_f/4$ seemed to reduce bond strength the most. The effect of defect was also more pronounced when the bond length ($L_f$) was much smaller than the effective bond length. In addition, for defects of the same size at location LE and FE, the bond strength reductions were not much different in most cases because the resulting bond areas were still similar.

Figure 8 Effect of defect on bond strength in models with varying $L_f$ (models with $f'_c = 23$ MPa, $b_c = 150$ mm, $b_f = 25$ mm, and $h_c = 5$ mm)

Figure 9 Effect of defect on bond strength in models with varying $b_f$ (models with $f'_c = 27.1$ MPa, $L_f = 100$ mm, $b_c = 150$ mm, and $h_c = 120$ mm)
Figure 9 shows the bond strength as affected by defect when the FRP width was varied, and the bond length was kept constant at 100 mm. Similar trends were observed, in which larger defects and defects at the center of bond joint resulted in higher bond strength reduction. In addition, higher percentage of bond strength reduction was observed in the models with wider FRP. This implied that the influence of defect was more pronounced in FRP/concrete bond joint with higher FRP stiffness.

CONCLUSIONS

This paper presents the derivation of stress distribution and bond strength in FRP/concrete bond joint with a defect by using finite element analysis. The FEA results have showed that characteristics of the defect, namely the location and size, influenced the interfacial shear stress distribution, the debonding behavior, and the bond strength. The effects of defect may be concluded as follows:

- Behavior of FRP/concrete bond joints with and without defect are similar in that they both consisted of three stages – elastic, elastic-softening, and elastic-softening-debonding, which depend on load level.
- Interfacial shear stress in FRP/concrete bond joint with defect was higher than that in FRP/concrete bond joint without defect under the same load due to lower true bond length.
- Larger defects resulted in higher interfacial shear stress, while defects in the zone of high shear stress (i.e. locations closer to the loaded end) could increase shear stress further.
- Bond strength of FRP/concrete bond joint was affected by the presence of defect. Larger defects resulted in higher reduction in bond strength, while defects in the zone of high shear stress (i.e. locations closer to the loaded end) can reduce bond strength further. The reduction in bond strength was more pronounced when the original bond length is much smaller than the effective bond length ($L_e$).
- FRP/concrete bond joint with higher FRP stiffness seemed to be affected more by the presence of defect.

ACKNOWLEDGEMENTS

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REFERENCES


EFFECT OF CONCRETE STRENGTH AND AMOUNT OF POLYUREA RESIN ON CONCRETE - STRAND SHEET BOND

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ABSTRACT

Bond performance of RC strengthened with polyurea resin and strand sheet depends on the concrete strength and the amount of polyurea resin. The present experimental study was undertaken in order to confirm the effect of above factors in the bond behaviour of concrete - strand sheet with a layer of polyurea resin by means of one side pull-out bond test method. Consequently, the specimen of the low strength concrete showed lower ultimate bond strength than that of the normal strength concrete at the same amount of polyurea resin regardless of presence or absence of the polyurea resin. However, the specimens with polyurea resin showed far higher ultimate strength than that of the specimens without polyurea resin regardless of the concrete strength. Furthermore, the bond strength was almost the same in the range of polyurea resin coating amount of 0.5-3kg/m².

KEYWORDS

Strand sheet, polyurea resin, concrete strength, bond, amount of polyurea resin.

INTRODUCTION

The effectiveness of flexural strengthening using the strand sheet is equivalent or greater compared to the one that uses the conventional fiber sheet, which has been shown experimentally (Akira et al. 2008). However, when peeling between the strand sheet and the concrete surface due to opening of flexure and shear cracks in concrete member occurs, the tensile strength of the strand sheet may not be provided properly. Improvement for this problem has been shown experimentally by inserting a layer of polyurea resin between the concrete and the strand sheet (Masaki et al. 2012). One of the reasons that polyurea resin was adopted is that the material properties of polyurea resin is more stable than those of other resins such as epoxy resin at a low temperature level of -20°C and at a high temperature level of 50°C. The bond tests under different temperature were conducted in order to confirm that polyurea resin is more stable at temperature. As a result, it was found that the certain effect was able to be obtained by using polyurea resin (Masaki and Akira et al. 2014, Masaki and Takahiro et al. 2014). However, how the concrete strength and the amount of polyurea resin may influence the bond behaviour had not yet been confirmed. Therefore, bond tests were carried out in order to confirm the impact of the factor, and the results have been reported.

BOND TEST PROGRAMS

Method of bond tests

The experimental report that the effective bond length of the specimens with polyurea is longer had been presented (Masaki et al. 2012). Therefore, one side pull-out bond test method was adapted due to constraints of the test equipment. Geometry of specimens is shown in Figure 1. Concrete blocks of size 150x300x600 mm were prepared. Specimens were 25mm in bond width and 520mm in bond length of the strand sheet and concrete. The release film was inserted at the upper tip of the concrete brock. During the loading tests, besides the load and the stroke...
of the test equipment (displacement), the strain of strand sheet was measured. Strain gauges were put in 40mm pitch from the end of the release film to measure the strain distribution.

![Diagram of Geometry of specimens](image)

**Figure 1 Geometry of specimens**

**Parameters of the bond tests**

As shown in Table 1, bond tests were carried out on different types of specimens in 9 cases, which include the case1 and the case5 without polyurea resin. Two or four specimens were prepared in each case. The experimental parameters were concrete strength and the amount of polyurea resin. Bond tests were selected at two different compressive strength of concrete (14.5N/mm² and 35.2N/mm² at the time of the experiment), in consideration of that some structures to apply this method have low strength concrete. Further, the coating amount of polyurea resin of the specimens was varied between 0-3kg/m², in consideration of that the concrete surface of the structure is not necessarily smooth although the standard application of the amount of polyurea resin on this method is 1.0kg/m². The outline of the construction process of the specimen is shown below. In Case1 and case5, strand sheet was attached using epoxy putty resin after grinding the concrete surface. In other specimens, urethane primer was coated at 0.2 kg/m² after grinding concrete surface. After polyurea resin was coated, the strand sheet was attached with epoxy putty resin. Properties of strand sheet and resin are shown in Table 2(a), 2(b).

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Concrete strength N/mm²</th>
<th>Amount of polyurea resin kg/m²</th>
<th>Number of specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>case1</td>
<td>35.2</td>
<td>Absence</td>
<td>2</td>
</tr>
<tr>
<td>case2</td>
<td>35.2</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>case3</td>
<td>35.2</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td>case4</td>
<td>35.2</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>case5</td>
<td>14.5</td>
<td>Absence</td>
<td>2</td>
</tr>
<tr>
<td>case6</td>
<td>14.5</td>
<td>0.5</td>
<td>2</td>
</tr>
<tr>
<td>case7</td>
<td>14.5</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>case8</td>
<td>14.5</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>case9</td>
<td>14.5</td>
<td>3</td>
<td>2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Tensile strength (MPa)</th>
<th>Elastic modulus (GPa)</th>
<th>Thickness (mm)</th>
<th>Fiber weight (g/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4520</td>
<td>259</td>
<td>0.333</td>
<td>600</td>
</tr>
</tbody>
</table>
Table 2(b) Material properties of resin

<table>
<thead>
<tr>
<th>Test items</th>
<th>Tensile strength (MPa)</th>
<th>Compressive strength (MPa)</th>
<th>Compressive elastic modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Urethane primer</td>
<td>Non-measurement</td>
<td>7</td>
<td>4</td>
</tr>
<tr>
<td>Polyurea resin</td>
<td>34</td>
<td>74</td>
<td>66</td>
</tr>
<tr>
<td>Epoxy resin putty</td>
<td>34</td>
<td>74</td>
<td>3930</td>
</tr>
</tbody>
</table>

RESULTS AND DISCUSSIONS

Maximum load and failure modes

Relationships of maximum load and amount of polyurea resin are shown in Figure 2. Maximum load and interfacial fracture energy are shown in Table 3. Interfacial fracture energy was calculated by using equation 1.

\[
G_f = \frac{P_{\text{max}}^2}{2 \cdot b^2 \cdot E_f \cdot t}
\]

where \(G_f\) is interfacial fracture energy [N/m], \(P_{\text{max}}\) is maximum load [N], \(b\) is strand sheet width [mm], \(E_f\) is elastic modulus of strand sheet [N/mm\(^2\)] and \(t\) is thickness of strand sheet [mm].

At the same amount of polyurea resin regardless of presence or absence of the polyurea resin, specimens of low concrete strength (14.5N/mm\(^2\)) showed approximately 0.75 times lower ultimate strength than that of the normal concrete strength (35.2N/mm\(^2\)). However, the specimens with polyurea resin regardless of amount of coating showed approximately 2.7 times higher ultimate strength than that of the specimens without polyurea resin also at the low-strength concrete. Interfacial fracture energy of the specimens with polyurea resin were approximately 7.0 times higher than that of the specimens without polyurea resin. Furthermore, the bond strength was almost the same in the range of polyurea coating amount of 0.5-3kg/m\(^2\). To stretch it a little, the specimens of 3kg/m\(^2\) showed slightly higher bond strength than that of the specimens of 0.5-2kg/m\(^2\).

Load - displacement curves and failure modes

Load - displacement curves are shown in Figure 3. It shows that the leaning of the load and displacement becomes moderate or load drops temporarily when strand sheet has partly peeled. Initial stiffness of the load displacement without polyurea resin is higher than that of the specimens with polyurea resin regardless of concrete strength. Initial stiffness of load and displacement was almost the same in the range of polyurea coating amount of 0.5-3kg/m\(^2\). To stretch it a little, the specimens of 3kg/m\(^2\) showed slightly lower initial stiffness of load and displacement than that of the specimens of 0.5-2kg/m\(^2\).

Typical failure modes of specimens are shown in Figure 4. The failure mode of all specimens was peeling failure of strand sheet. All specimens without polyurea resin showed the failure modes that concrete was thinly adhered to the strand sheet side such as in case1-1. Some specimens with polyurea resin showed the failure modes that concrete was thickly adhered to the strand sheet side such as in case3-2 in comparison with specimens without polyurea resin. Others showed partly cohesive failure of polyurea resin such as in case2-2.

Figure 2 Relationships of maximum load and amount of polyurea resin
Table 3 Summary of test results

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Maximum load (kN)</th>
<th>Interfacial fracture energy (Gf) (N/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Data Average</td>
<td>Data Average</td>
</tr>
<tr>
<td>case1-1</td>
<td>9.5 9.3</td>
<td>0.84 0.76</td>
</tr>
<tr>
<td>case1-2</td>
<td>9.0</td>
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</tr>
<tr>
<td>case2-1</td>
<td>21.0</td>
<td>4.10</td>
</tr>
<tr>
<td>case2-2</td>
<td>27.0</td>
<td>6.76</td>
</tr>
<tr>
<td>case2-3</td>
<td>17.7</td>
<td>2.91</td>
</tr>
<tr>
<td>case2-4</td>
<td>27.2</td>
<td>6.86</td>
</tr>
<tr>
<td>case3-1</td>
<td>22.8</td>
<td>4.82</td>
</tr>
<tr>
<td>case3-2</td>
<td>26.5</td>
<td>6.51</td>
</tr>
<tr>
<td>case3-3</td>
<td>22.7</td>
<td>4.78</td>
</tr>
<tr>
<td>case3-4</td>
<td>21.8</td>
<td>4.39</td>
</tr>
<tr>
<td>case4-1</td>
<td>27.3</td>
<td>6.91</td>
</tr>
<tr>
<td>case4-2</td>
<td>29.1</td>
<td>7.85</td>
</tr>
<tr>
<td>case5-1</td>
<td>8.5</td>
<td>0.67</td>
</tr>
<tr>
<td>case5-2</td>
<td>5.2</td>
<td>0.25</td>
</tr>
<tr>
<td>case6-1</td>
<td>17.8</td>
<td>2.95</td>
</tr>
<tr>
<td>case6-2</td>
<td>19.4</td>
<td>3.48</td>
</tr>
<tr>
<td>case7-1</td>
<td>15.3</td>
<td>2.16</td>
</tr>
<tr>
<td>case7-2</td>
<td>18.0</td>
<td>3.01</td>
</tr>
<tr>
<td>case8-1</td>
<td>16.4</td>
<td>2.49</td>
</tr>
<tr>
<td>case8-2</td>
<td>18.9</td>
<td>3.31</td>
</tr>
<tr>
<td>case9-1</td>
<td>18.4</td>
<td>3.13</td>
</tr>
<tr>
<td>case9-2</td>
<td>22.1</td>
<td>4.54</td>
</tr>
</tbody>
</table>

(a) concrete strength 35.2N/mm²  (b) concrete strength 14.5N/mm²

Figure 3 Load - displacement curves

Figure 4 Typical failure modes of specimens
Strain distribution

Strain distributions at the maximum load are shown in Figure 5. In Case1-1 and case5-1 without polyurea resin, FRP strains have locally changed at certain locations regardless of concrete strength. For example, FRP strain of Case1-1 varied from 40 mm to 160 mm of location. Thus, it was found that bond stress was shared in almost 100 mm length. FRP strains of other specimens with polyurea resin have gradually changed in comparison with the specimens without polyurea resin. Therefore, it was found that bond stress was shared for longer length because polyurea resin has low Young’s modulus and high elongation. Because of this characteristic, the specimens with polyurea resin are able to have higher ultimate strength than the specimens without polyurea resin. In addition, the more the amount of the polyurea resin coat, the more the range of bond stress spread regardless of concrete strength.

(a) concrete strength 35.2N/mm²
(b) concrete strength 14.5N/mm²

Figure 5 Strain distributions at the maximum load

Bond stress and effective bond length

Relationships between bond stress or effective bond length and amount of resin at the maximum load are shown in Figure 6. Bond stress and effective bond length were calculated by using equation 2, 3 (JCI publication).

\[
\tau_y = \frac{\Delta \varepsilon_f E_F A_F}{s_i b} \quad \text{(2)}
\]

\[
l_e = \frac{P}{\tau_y b} \quad \text{(3)}
\]

where \(\tau_y\) is bond stress [N/mm²], \(\Delta \varepsilon_f\) is the difference of the value of the strain in the strain increase section, \(E_F\) is elastic modulus of strand sheet [N/mm²], \(A_F\) is cross-sectional area of strand sheet [mm²], \(s_i\) is distance of the strain increase section [mm], \(b =\) strand sheet width [mm] and \(P\) is load [kN].

Regardless of the concrete strength, bond stress of specimens with polyurea resin was smaller than that of the specimens without polyurea resin. However, effective bond length of specimens with polyurea resin was about 5 times longer than that of the specimens without polyurea resin. By this long effective bond length, the specimens
with polyurea resin are able to have higher ultimate strength than that of the specimens without polyurea resin. On the specimens with polyurea resin, the more amount of the polyurea resin coat, the shorter the bond stress is. In the case of the same amount of polyurea resin, the specimens of the low strength concrete showed slightly lower bond stress and shorter effective bond length than that of the normal strength concrete. By low bond stress and this short effective bond length, the specimens of low strength concrete with polyurea resin have lower ultimate strength than the specimens of normal strength concrete with polyurea resin.

CONCLUSIONS

1. The specimens of low concrete strength showed lower ultimate strength than that of the normal concrete strength at the same amount of polyurea resin regardless of presence or absence of the polyurea resin. However, the specimens with polyurea resin regardless of amount of coating showed approximately 2.7 times higher ultimate strength than that of the specimens without polyurea resin also at the low strength concrete. Interfacial fracture energy of the specimens with polyurea resin were approximately 7.0 times higher than that of the specimens without polyurea. Furthermore, the bond strength was almost the same in the range of polyurea coating amount of 0.5-3kg/m².

2. The failure mode of all specimens was peeling failure of strand sheet. All specimens without polyurea resin showed the failure modes that concrete was thinly adhered to the strand sheet side. Some specimens with polyurea resin showed the failure modes that concrete was thickly adhered to the strand sheet side in comparison with the specimens without polyurea resin. Others were partly cohesive failure of polyurea resin.

3. Regardless of concrete strength, bond stress of specimens with polyurea resin was smaller than that of the specimens without polyurea resin. However, effective bond length of specimens with polyurea resin was about 5 times longer than that of the specimens without polyurea resin. By this long effective bond length, the specimens with polyurea resin have higher ultimate strength than that of the specimens without polyurea resin.

As a result, it was proved that the polyurea resin was effective for improving bond ability between the strand sheet and concrete surface at low strength concrete as well as normal strength concrete and in the range of polyurea resin coating amount of 0.5-3kg/m².

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ABSTRACT

In the construction field composite materials as a strengthening are widely used to face the problem of ageing and overloading of old infrastructure. The material performances are often deteriorated or inadequate for the current loads and could be compared to nowadays low resistance materials. The here presented research aims at analysing the debonding process on concrete weak resistance classes reinforced with externally glued CFRP strips. The joint behaviour is investigated with single-shear tests technique varying concrete performances and gluing techniques. The debonding growth are compared considering a dimensionless parameter dependent on the fracture energy. The results point out different behaviours among the concrete classes. The characterization of the cracking patterns is analysed with specific roughness measurements offering a clear link to the fracture energy of the debonding process. This finding is opening new possibilities to bond models development.

KEYWORDS

CFRP, concrete roughness, strengthening, bond.

INTRODUCTION

For several decades carbon fibres have been used to retrofit ancient building or to extend the service life of infrastructures. In the past years the research focused on the study of the interface law between fibres and substrate, the maximal force and the durability of the joint. Bond failure is a brittle phenomenon caused by stress accumulation on the joint interface and occurs without any warning (Täljsten 1995; Triantafillou, Plevris 1992). One of the main purposes of this study is to predict bond resistance as the conditions of the material change. The research aims at finding the parameters that influence the behaviour of the joint and therefore the bond failure. Different aspects can affect the behaviour of the joint; among others we can mention geometrical characteristics (width of the strips related to the width of reinforced elements), peculiarity of material and treatments like surface preparation (Iovinella et al. 2013; Mazzotti et al. 2009).

The paper presents an experimental investigation on the behaviour of the bond varying gluing techniques and mechanical properties of the substrate. The debonding phenomenon was analysed in a continuous displacements field using the digital image correlation technology.

The study considers the single shear test results, as well as the behaviour of fracture propagation in different test configurations. A characterisation of the fracture plane is conducted analysing the roughness of surfaces for the two materials after the tests. In order to compare the different fracture patterns, we took into account several roughness parameters, measured with different instruments and methods.

OBJECTIVES

The goal of this paper is investigating how the substrate performance and the gluing techniques affect the fracture pattern. The carbon fibre-concrete bond was proved with a single-shear test setup. The experimental campaign involved concrete specimen with different mixture (several aggregate dimensions) and reinforcing gluing techniques. Particular importance took the roughness study of the surfaces of strips and concrete after reinforcing comparing different methods and parameter. The calculated roughness values seem to present correlation with the energy release of the debonding.

EXPERIMENTAL SET UP
The test program was divided in two series, where the concrete mixture and gluing techniques were varied. In the first series a parameter study on the concrete mixtures and gluing techniques was conducted. Two different concrete mixtures (8 and 16mm maximal aggregate dimension) were prepared in order to obtain performances comparable to strength class C16/20 and C20/25 according to Eurocode 2 (EN 1992-1-1). Moreover, two different gluing techniques were used: a strip set was glued using only epoxy resin; for the other one a primer film was spread on the surface before gluing.

Furthermore, to analyse the effect of the primer for lower strength concrete classes, another specimen series (C12/15) was casted using the same two gluing techniques applied in the previous series. For ease of reference in the further tables the test groups are mentioned with their own resistance class. If the word “Primer” displays it means that the CFRP is glued in combination with the primer. As concrete support, prisms with 50 cm height and 15 cm square section similar to (Bizindavyi, Neale 1999) were used. All the specimens were unmoulded 24 hours after casting, submerged in water for 7 days and stored under norm climate 20º C and 65% r.h. for the next 21 days.

The compression and traction values for the three mixtures are presented in the Tables 1 and 2.

<table>
<thead>
<tr>
<th>Concrete Class</th>
<th>$f_{cm}$ [MPa]</th>
<th>Standard deviation [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>C16/20</td>
<td>31.5</td>
<td>2.72</td>
</tr>
<tr>
<td>C20/25</td>
<td>38.21</td>
<td>2.28</td>
</tr>
<tr>
<td>C12/15</td>
<td>17.88</td>
<td>1.19</td>
</tr>
</tbody>
</table>

Before gluing the CFRP strips, the surface of all the concrete prisms were treated with a grinding machine and then cleaned with pressured air in order to remove the superficial layer (2-3 mm). Considering different norms and guidelines recommendation (CNR-DT 200), (ACI 440.2R-08) a 35 cm bond length was adopted. The glued length was taken 10 cm away from the loaded edge to avoid effects like wedge cracking. The pre-impregnated carbon strips present a constant section dimensions with 50 mm width and 1.4 mm thickness.

To evaluate the bond behaviour the near-end supported single-shear test setup (Yao et al. 2005) was adopted. The slip evolution during the test was represented with a digital image correlation system “Aramis” (DIC) furnished by GOM and composed of two 5 Megapixel cameras (Optical measurement system). The two cameras take pictures following a trigger set synchronized with the displacement increments of the proof machine. Beside the digital image correlation measurements, two strain gauges were placed respectively at 5 cm and 13 cm to the upper edge of the concrete specimen, i.e. on the unglued and on the glued strip portion.

The tests were carried out following a monotonic slip increase till the failure; the slip constant rate was 0.01 mm/s. The restrain system was made of a screw anchored on a bottom steel plate passing in to the concrete hollow and a piston located on the top of the specimen. To avoid peeling stresses, the concrete specimens were prestressed before applying the force at the strip. To distribute the force on the concrete specimen a 2 cm thick steel plate was placed between the piston and the concrete. The test set up is represented in Figure 1.

**EXPERIMENTAL RESULT**

The test results in terms of maximal force registered with the testing machine are represented in Table 3. For every specimen we calculated the fracture energy related to the square root of concrete tensile and compressive strength Table 3. The values of the fracture energy coefficient increases for the low resistance classes; this trend seems to lead to another failure mechanism that could be influenced with a primer film before gluing.

The fracture coefficient was afterwards represented in a diagram with the roughness parameters in order to find a correlation between the values.

In order to obtain the values of the shear stress and slip we adopted the bi-linear slip-stress law (Holzenkämpfer, 1994). The strains were obtained comparing the slip values from the DIC measurements and the strain gauges. The slip from DIC were calculated on the incipient debonding zone, in order to avoid the scattering of values caused of singularities we considered the average value on three sections.
All the debonding failures occurred on the concrete substrate. Looking at the slip-force diagrams no significant effect of the primer in the bond displays; the behaviour was similar also between different concrete mixtures. In the second series (C12/15) a greater difference arises. The primer influence seems to have a considerable effect on lower concrete mixture. The maximal registered force for the two groups have a minimum range value of circa 4 kN. The effect of the primer seems to be related to the peculiarity of the primer to percolate deeper in the concrete substrate.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximal force [kN]</th>
<th>$G_F \sqrt{f_{cm} * f_{ctm}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>C16/20 Primer</td>
<td>28,851</td>
<td>0.067</td>
</tr>
<tr>
<td>C16/20 Primer</td>
<td>26,885</td>
<td>0.109</td>
</tr>
<tr>
<td>C16/20</td>
<td>28,368</td>
<td>0.114</td>
</tr>
<tr>
<td>C16/20 Primer</td>
<td>23,973</td>
<td>0.096</td>
</tr>
<tr>
<td>C20/25 Primer</td>
<td>24.2</td>
<td>0.069</td>
</tr>
<tr>
<td>C20/25 Primer</td>
<td>25,082</td>
<td>0.078</td>
</tr>
<tr>
<td>C20/25</td>
<td>31,052</td>
<td>0.063</td>
</tr>
<tr>
<td>C20/25 Primer</td>
<td>27,071</td>
<td>0.101</td>
</tr>
<tr>
<td>C12/15 Primer</td>
<td>27,954</td>
<td>0.293</td>
</tr>
<tr>
<td>C12/15 Primer</td>
<td>23.880</td>
<td>0.069</td>
</tr>
<tr>
<td>C12/15 Primer</td>
<td>24.261</td>
<td>0.195</td>
</tr>
<tr>
<td>C12/15</td>
<td>19,841</td>
<td>0.206</td>
</tr>
<tr>
<td>C12/15 Primer</td>
<td>19,682</td>
<td>0.232</td>
</tr>
<tr>
<td>C12/15 Primer</td>
<td>19,722</td>
<td>0.179</td>
</tr>
</tbody>
</table>

Table 3 Maximal bonding force for the two test series with the relative fracture energy dependent parameter in the right column

**ROUGHNESS MEASUREMENTS**

An important part of this experimental investigation is the study of the fracture patterns on the carbon fibres strip and on the concrete substrate, i.e. after debonding. In order to compare the different fracture patterns, the roughness of the strips and the concrete substrate were measured with different methods and parameter. The roughness parameters considered are available on norms and scientific publications (Santos, Júlio 2013), the data were collected with an Optical Measurements System (previously used with as DIC for the shear test) and an Optical Microscope.

Thanks to flexibility of the Optical Measurement system “Aramis” both fracture surfaces, on the CFRP strip and also at the concrete substrate, where digitized. The measurement provides a point mesh with a mean distance between two point of circa 0.08 mm.
The Keyence VHX-2000 Microscope provides a tridimensional mesh point description of the analysed surfaces with a resolution of circa 1 µm between two point. For physical reasons it was only possible to collect the data for the CFRP strip surface. The measurements were focused on the incipient debonding area, i.e. the area where the strips are pulled out tearing off a substrate of the concrete.

To better compare all the parameters collected during the research the parameters based on 2D profiles were considered. This choice is due to the fact that the available parameters are most of the times defined for two dimensional profiles.

ROUGHNESS PARAMETERS

The most important roughness parameters in the analysis are four: two from European norms and two from scientific publications. Referring to the European Norms ISO 4287 the following parameters are considered: the average roughness \( R_a \), which represents the absolute mean value of the deviation between the real surface and the mean value, and the root mean square \( R_q \) of the profile, which is the mean square value between the real value and the mean line. The two parameters are here presented:

\[
R_a = \frac{1}{l} \int_0^l |Z(x)| \, dx \approx \frac{1}{n} \sum_{i=1}^n |Z_i|; \quad R_q = \sqrt{\frac{1}{l} \int_0^l |Z(x)|^2 \, dx} \approx \sqrt{\frac{1}{n} \sum_{i=1}^n |Z_i|^2}
\]  

(1)

Where \( n \) is the measurements number and \( Z_i \) is the data of the point magnitude.

Another important parameter was for the first time introduced from (Iovinella et al., 2013) and has the following expression:

\[
I_R = R \cdot i_A
\]

(2)

It considers the product of two terms, \( R \) that is the amplitude of the profile valleys and \( i_A \) that is the inclination between the points (angularity).

Already mentioned in other researches is the Wenzel coefficient to characterize the concrete surfaces describing the ratio between the profile’s and its effective length:

\[
W = \frac{1}{l} \sum_{i=1}^n \sqrt{(\Delta x_i)^2 + (\Delta y_i)^2}
\]

(3)

where \( l \) is the profile length and \( \Delta x \) and \( \Delta y \) are respectively the increment in vertical and horizontal length between two points of the profile.

All the data collected were elaborated with the same procedure in a Matlab code. For every surface significant profiles were selected, the inevitable inclination of the measurement obtained from optical and microscope measurement was corrected and all the previously mentioned roughness parameter were calculated. To have a brighter profile parameter distribution, the roughness parameters were calculated from several profiles and the average of the values was assumed.

The values obtained for all the concrete and CFRP surfaces observed are presented respectively in Table 4 and 5.

The parameters values are not significantly influenced on the measuring techniques. The difference in the fracture surface roughness as an effect of primer preparation increases for the low performance concrete specimens on the CFRP strip and on the concrete substrate as well. For the higher concrete strength classes, the primer preparation has no effect on the fracture roughness. The results obtained are presented in the histograms for the strip surfaces in terms of \( R_a \) for the data collected with the optical measurement in Figure 5. For every roughness parameter the respective dimensionless values were plotted. In Figure 6 a diagram with the roughness parameter correlated with the related fracture energy is shown. A good correlation between these two parameters seems to be confirmed by the coefficient of determination values although the small number of tests conducted (Table 6).
<table>
<thead>
<tr>
<th>Table 4.5 Roughness parameter from the optical measurements.</th>
<th>(a) Concrete surfaces values</th>
<th>(b) CFRP surfaces values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$R_a$ [mm]</td>
<td>$R_q$ [mm]</td>
</tr>
<tr>
<td>C16/20 Primer</td>
<td>0.5119</td>
<td>0.6246</td>
</tr>
<tr>
<td>C16/20 Primer</td>
<td>0.5691</td>
<td>0.699</td>
</tr>
<tr>
<td>C16/20</td>
<td>0.5445</td>
<td>0.6954</td>
</tr>
<tr>
<td>C16/20</td>
<td>0.4213</td>
<td>0.5294</td>
</tr>
<tr>
<td>C20/25 Primer</td>
<td>0.4179</td>
<td>0.5123</td>
</tr>
<tr>
<td>C20/25 Primer</td>
<td>0.4234</td>
<td>0.529</td>
</tr>
<tr>
<td>C20/25</td>
<td>0.574</td>
<td>0.7166</td>
</tr>
<tr>
<td>C20/25</td>
<td>0.5381</td>
<td>0.666</td>
</tr>
<tr>
<td>C12/15 Primer</td>
<td>1.1818</td>
<td>1.439</td>
</tr>
<tr>
<td>C12/15 Primer</td>
<td>1.1066</td>
<td>1.3523</td>
</tr>
<tr>
<td>C12/15 Primer</td>
<td>0.9987</td>
<td>1.1943</td>
</tr>
<tr>
<td>C12/15</td>
<td>0.7269</td>
<td>0.8945</td>
</tr>
<tr>
<td>C12/15</td>
<td>0.828</td>
<td>1.0197</td>
</tr>
<tr>
<td>C12/15</td>
<td>0.6112</td>
<td>0.7617</td>
</tr>
</tbody>
</table>

Figure 5 Histogram of the $R_a$ parameters for the fiber strips from microscope measurements.  
Figure 6 Energy dependent parameter-Wenzel coefficient graph with linear interpolation.

Table 6 Coefficients of the determination of the energy dependent coefficient/roughness parameters.

<table>
<thead>
<tr>
<th>Device</th>
<th>Surface</th>
<th>Parameter</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Optical meas.</td>
<td>Concrete</td>
<td>$R_a$</td>
<td>0.845</td>
</tr>
<tr>
<td>Optical meas.</td>
<td>CFRP</td>
<td>$R_a$</td>
<td>0.757</td>
</tr>
<tr>
<td>Optical meas.</td>
<td>Concrete</td>
<td>Wenzel coeff.</td>
<td>0.842</td>
</tr>
<tr>
<td>Optical meas.</td>
<td>CFRP</td>
<td>Wenzel coeff.</td>
<td>0.794</td>
</tr>
<tr>
<td>Optical meas.</td>
<td>Concrete</td>
<td>$I_R$</td>
<td>0.878</td>
</tr>
<tr>
<td>Optical meas.</td>
<td>CFRP</td>
<td>$I_R$</td>
<td>0.817</td>
</tr>
<tr>
<td>Optical meas.</td>
<td>Concrete</td>
<td>$R_q$</td>
<td>0.856</td>
</tr>
<tr>
<td>Optical meas.</td>
<td>CFRP</td>
<td>$R_q$</td>
<td>0.476</td>
</tr>
<tr>
<td>Microscope</td>
<td>CFRP</td>
<td>$R_q$</td>
<td>0.806</td>
</tr>
<tr>
<td>Microscope</td>
<td>CFRP</td>
<td>Wenzel coeff.</td>
<td>0.771</td>
</tr>
<tr>
<td>Microscope</td>
<td>CFRP</td>
<td>$I_R$</td>
<td>0.813</td>
</tr>
</tbody>
</table>

CONCLUSIONS

The paper presents an experimental investigation of the bond for the carbon fibres glued on concrete support. The analysis aims at representing the fracture pattern after the single-shear test in different gluing and concrete mixture configuration to proof different debonding evolution. To represent the propagation of the debonding on the
concrete substrate several roughness parameters, determined with different measurement instruments, were considered. A linear relation shows a good correlation between the dimensionless fracture energy coefficient and the roughness parameters. It arises that a surface preparation with a primer can have the same effect as an intensive mechanical surface preparation.

The influence of the gluing techniques, especially for low performances concrete support, seems to be significant. This effect put in evidence a different debonding behaviour for this kind of concrete. The study of weak concrete performance assumes a great importance because they can be compared to the aged structures the carbon fibre-reinforcements are applied on. The results obtained in this experimental investigation reveal a different fracture behaviour for the low performances concrete and open the possibility of further experimental campaigns to better understand the debonding for these configurations.

REFERENCES

DIN EN ISO 4287: Oberflächenbeschaffenheit: Tastschnittverfahren, Benennung, Definition und Kenngrößen, July 2010;
EXPERIMENTAL STUDY ON BOND BEHAVIOR OF EXTERNALLY BONDED BFRP SHEETS - CONCRETE JOINTS UNDER SUSTAINED LOADING

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5 Tsuchiya TSCO Co., Ltd., Japan.

ABSTRACT

The FRP debonding due to the shear stress distinguished around flexural cracks on the RC or PC structure strengthened with externally bonded FRP sheets has been of recent concern. In a previous study, the FRP debonding progresses rapidly under continuous loading. In this study, a tension test of an FRP-concrete double lap shear adhesive joint under sustained loading has been performed. A basalt fibre reinforced polymer (BFRP) sheet was used as the FRP reinforcement. An increase of the pre-crack width and distributed BFRP strain under sustained loading with time have been identified. The load was set at approximately 60% of the debonding load under static loading. From the results, it was observed that the shear stress can be distinguished at the area from the pre-crack to the effective bond length, the stress was greater than the concrete shear strength, and a slip into the concrete surface layer occurred. It was also found that the BFRP debonding slightly propagated until approximately 100 hours from the load being applied, with no significant debonding from 100 to 1000 hours.

KEYWORDS

Basalt FRP (BFRP) sheets, externally bond, concrete, double-lap shear test, distributed strain, sustained load.

INTRODUCTION

RC or PC structures in service have an increased dead load that results from structural changes in consideration of the performance requirements or from the damage of concrete reinforcements due to corrosion from a long service life. As a result, a shear stress distributed into the FRP sheets around the flexural cracks has increased, and a premature debonding of the FRP sheet is of concern. In a previous study, it was found that premature debonding occurred due to a continuous load (Iwashita et al. 2007). On the other hand, the maximum upper limit load, 2 millions of cyclic loading reachable, was half of the static maximum load (lower limit load is 10% of the static maximum load). The sustained load has a significant influence on the lifetime of the FRP-concrete bonding joints. From this circumstance, investigating the behaviour of the BFRP-concrete interface under sustained loading is important. On the other hand, basalt fibre reinforced polymer (BFRP) sheets have the excellent properties of ductility, strength and modulus of elasticity. It has been attempted to utilize BFRPs in practical situation as a seismic reinforcement for old type or determined RC and PC structures. In this study, BFRP was used as an FRP reinforcement.

TEST SET-UP

Specimen of BFRP-concrete Double-lap Joints

The dimensions of the specimens are shown in Fig. 1. In accordance with the experimental method of the JSCE Recommendations for Upgrading of Concrete Structures with Use of Continuous Fibre Sheets (JSCE, 2000), the specimen was manufactured with two concrete prisms of a 100 mm width, 100 mm height and 300 mm length and was externally bonded to two BFRP sheets with a thickness of 0.254 mm and a width of 50 mm. A summary of the properties of the BFRP sheets, epoxy resin and concrete are shown in Table 1. On the double-lap shear joints, the effects from a flexural moment can be eliminated, unlike the single-lap shear joints. The specimen has a slit (pre-crack) simulating a flexural crack in the concrete at the mid-span. First, the concrete surfaces were treated with a diamond sander, wiped with an acetone soaked cloth and painted with a room temperature setting epoxy
primer adding a total mass of 0.5 kg/m². The sample was cured at room temperature for twelve hours later. After curing, a single BFRP sheet was impregnated and bonded to the concrete surface on both sides along the axial direction with a room temperature setting epoxy resin with a total mass of 0.5 kg/m² for each. The sample was cured at room temperature for seven days, and the tensile sustained test was started.

**Figure 1** Detail of double-lap shear specimen

**Table 1** Summary of material properties

<table>
<thead>
<tr>
<th>Materials</th>
<th>Characteristics</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>BFRP sheets</strong></td>
<td>Tensile strength $\sigma_f$ (N/mm²)</td>
<td>1,900</td>
</tr>
<tr>
<td></td>
<td>Tensile modulus of elasticity $E_f$ (N/mm²)</td>
<td>90,000</td>
</tr>
<tr>
<td></td>
<td>Rupture strain (µ)</td>
<td>21,111</td>
</tr>
<tr>
<td></td>
<td>Nominal thickness $t_f$ (mm/1 layer)</td>
<td>0.254</td>
</tr>
<tr>
<td></td>
<td>Fibre volume $V_f$ (%)</td>
<td>45-50</td>
</tr>
<tr>
<td><strong>Epoxy resin</strong></td>
<td>Tensile strength $\sigma_e$ (N/mm²)</td>
<td>45</td>
</tr>
<tr>
<td><strong>Concrete</strong></td>
<td>Compressive strength (N/mm²)</td>
<td>37.3</td>
</tr>
</tbody>
</table>

**Experimental Programme and Measurement**

A tensile load was applied from the connected steel bolts at both ends of the steel frame test device by pulling in both directions with a 350kN load cell and a 200kN hydraulic jack and pump as shown in Fig. 2. The 25 mm displacement sensor was set to the side of the specimen near the pre-crack to measure the expansion of pre-crack width and 5 mm strain gauges were set to the BFRP surface at 25 mm distances from the pre-crack to measure the BFRP strain distribution. The arrangements are shown in Fig. 1.

The established creep load was 25kN and the interfacial fracture energy ($G_f$) signifying the bond capacity, was calculated from Eq. 1;

$$G_f = \frac{P_{\text{max}}^2}{8b_fE_f t_f}$$

where $P_{\text{max}}$ is the maximum load and $E_f$, $b_f$ and $t_f$ are the modulus of elasticity, width and thickness of the BFRP sheets respectively. The calculated $G_f$ is 1.37 N/mm, which is equivalent to the maximum value in the static tests.
from a previous study (Iwashita et al. 2014). The loading rate was 5kN/min until a maximum of 25 kN was reached. The test temperature was approximately 12-16°C.

TEST RESULTS AND DISCUSSION

Debonding Propagation

The average BFRP strain at the pre-crack under a sustained load was 10,936 μ. The strain is significant and equal to the debonding strain. The external appearance of the BFRP surface after 15 hours is shown in Fig. 3. In the area from the pre-crack to 75 mm, several cracks appeared on the FRP surface, and the premature debonding was confirmed to the area with hammering test. It is considered that these cracks and premature debonding occurred due to the shear stress concentration at the pre-crack. The evaluation Eq. 2a to Eq. 2c of an effective bond length ($L_e$) were suggested (ACI Committee 440 2008, Sato et al. 2000, Wu et al. 2007).

$$L_e = n \cdot \frac{E_f \cdot t_f}{f_c'}$$  \hspace{1cm} (2a)

$$L_e = 1.89(t_f \cdot E_f)^{0.4}$$ \hspace{1cm} (2b)

$$L_e = \frac{1.3}{f_c' \cdot 0.095}$$ \hspace{1cm} (2c)

where $n$ is the number of the BFRP layers and $f'_c$ is the compressive strength of the concrete. Calculation values are 61 mm (by Eq. 2a), 105 mm (by Eq. 2b) and 139 mm (by Eq. 2c). It is observed that the shear stress was distinguished in the $L_e$, the stress exceeded the concrete shear strength and a slip into the concrete surface layer occurred.

Applied Load Accuracy

The applied load degradation with time are shown in Figs. 4 (a) – (d) for each time span. From 0 - 500 hours from the load being applied, the load larger than the setting one of 25 kN. The sustained load was variable in the range of ± 5 kN.
Pre-crack Width Expanding with Time

The expansion of the pre-crack has a function of time are shown in Figs. 5 (a) - (d) for each time span. From 0 - 250 hours from the load being applied, the width increased logarithmically. The rate of increase became increasingly smaller after 250 hours and even smaller after 500 hours. The BFRP strain distribution degradation with time are shown in Figs. 6 (a) - (d) for each time span. The strain was distinguished at 125 mm from the pre-crack from 0.01 - 0.1 hours, and it was found that the BFRP debonding propagated 125 mm. From 0.01 - 100 hours from the load being applied, the distributed BFRP strain increased gradually with time. However, after 100 hours, the strain had a negligible increase.
CONCLUSIONS

This paper has experimentally investigated the change of the distributed strain of the externally bonded BFRP sheets to concrete and pre-crack under sustained loading. Based on the results of this study, the following conclusions are drawn:

1) The shear stress was distinguished in the effective shear transfer region and exceeded the concrete shear strength. The premature debonding of the BFRP sheets occurred in the area, but the incremental strain was relatively small under sustained loading.

2) The incremental pre-crack width did not significantly change under the sustained loading.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the financial support provided by the Advanced Research Center for Natural Disaster Risk Reduction (NDRR) at Meijo university, Japan.
Figure 6 The BFRP strain distribution degradation with time.

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ACI Committee 440 (2008), “Guide for the design and construction of externally bonded FRP systems for Strengthening Concrete structure (ACI 440.2R-8)”.

EXTRACTION OF BOND-SLIP CHARACTERISTICS IN FRP-TO-CONCRETE JOINTS EXPOSED TO ACCELERATED AGGRESSIVE ENVIRONMENTS

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ABSTRACT

Strengthening of structurally deficient reinforced concrete elements by means of adhesively bonding fibre-reinforced polymers (FRP) is now common practice. Given the uncertainty of the long-term durability of these strengthening systems, the critical bond between the FRP and concrete has seen widespread testing of samples conditioned under accelerated aggressive environments. In order to develop a set of bond deterioration factors, a database of test results containing full range load-displacement relations of FRP-to-concrete joints under various environmental conditions has been collected and a mechanics based approach has been shown to be able to extract bond properties across the various test set-ups, material properties and environmental exposures. Through a statistical analysis, conservative deterioration factors are determined that allow for the design of future durability bond tests that can ensure that the full range load-displacement behaviour is captured.

KEYWORDS

FRP, concrete, bond, durability, effective bond length.

INTRODUCTION

The structural strengthening of reinforced concrete (RC) members by means of adhesively bonding fibre-reinforced polymers (FRP) is now common practice. However, there is uncertainty surrounding the long-term durability of the strengthened system. Current guidelines are only able to demonstrate reservations for environmental effects through a reduction factor of the rupture elongation of FRP in the order of 0.85-0.95 (ACI Committee 440 2008) or imposing a maximum design life (Concrete Society Working Party 2012). Moreover, and unlike conventional RC members that fail by yielding of the steel reinforcement followed by concrete crushing, FRP-strengthened beams are susceptible to premature debonding failures that can severely limit performance. These have been attributed to stress concentrations at the plate end, widening of flexure-shear cracks and through the IC debonding mechanism associated with the widening of flexural cracks (Oehlers and Moran 1990). Design guidelines reflect this by assuming a perfect bond exists between the FRP and concrete and limiting the strain in the FRP. However, the ability of the joint to the transfer stresses between FRP and concrete has not only implications for flexural strength, but also ductility (Oehlers et al. 2016). Shear bond tests isolate the conditions at flexural cracks that cause IC debonding and investigate bond behavior to not only aid in the design of strengthened members, but allow researchers to model the full structural response.

Concerns over the durability of the bond between FRP and concrete has prompted widespread shear bond testing of joints conditioned under various accelerated aggressive environments. A previous study by the authors (Aydin et al. 2016) identified 1,169 individual samples form 48 separate studies to assess environmental impacts on shear bond strength. Despite widespread laboratory testing, reporting of the load-displacement, P-δ relationship, necessary for modelling the full range structural response and deriving full range bond stress-slip, τ-δ relationships are found to be limited to the studies summarised in Table 1 amounting to 165 individual tests. Using these P-δ responses from the published literature, a well-established numerical method (Haskett et al. 2008) is used to extract τ-δ relationships. The results are analyzed to determine bond deterioration factors for a range of conditions (Gravina et al. 2016).
Table 1 Durability tests including full range load-displacement relationships

<table>
<thead>
<tr>
<th>Reference</th>
<th>Test conditions^a</th>
<th>Test setup^b</th>
<th>Type of FRP^c</th>
<th>Failure modes^d</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mukhopadhyaya et al. (1998)</td>
<td>Wet-dry cycling in 5% NaCl, room temperature, 18 cycles</td>
<td>DLS</td>
<td>GFRP plate</td>
<td>C, A</td>
</tr>
<tr>
<td>Wu et al. (2004)</td>
<td>Elevated temperature 26 to 50 °C, 6.5 hours</td>
<td>DLS</td>
<td>CFRP sheet</td>
<td>P</td>
</tr>
<tr>
<td>Au and Büyükoztürk (2006)</td>
<td>Continuous immersion in water, 23 to 50 °C, 8 weeks</td>
<td>SLS</td>
<td>CFRP plate</td>
<td>A</td>
</tr>
<tr>
<td>Fava et al. (2007)</td>
<td>Salt fog 5% NaCl, 50 °C, 1 month</td>
<td>SLS</td>
<td>CFRP plate</td>
<td>NR</td>
</tr>
<tr>
<td>Tam (2007)</td>
<td>Freeze-thaw cycling, -20 to 20 °C, 300 cycles, sustained loading</td>
<td>DLS</td>
<td>CFRP plate</td>
<td>C</td>
</tr>
<tr>
<td>Klammer (2009)</td>
<td>-20 to 100 °C, 16 hours</td>
<td>DLS</td>
<td>CFRP plate</td>
<td>C, A</td>
</tr>
<tr>
<td>Yun and Wu (2011)</td>
<td>Freeze-thaw in water and 4% NaCl solution, -17 to 4 °C, up to 67 cycles, sustained loading</td>
<td>SLS</td>
<td>CFRP sheet</td>
<td>C</td>
</tr>
<tr>
<td>Shi et al. (2013)</td>
<td>Freeze-thaw cycling, -17 to 4 °C, 300 cycles</td>
<td>DLS</td>
<td>BFRP sheet</td>
<td>C, A</td>
</tr>
<tr>
<td>Gravina et al. (2016)</td>
<td>Continuous immersion in water, 5% NaCl and pH 4.0 sulphuric acid solution, 22 °C, 8 weeks</td>
<td>SLS</td>
<td>CFRP plate</td>
<td>C, A, AF</td>
</tr>
</tbody>
</table>

Note:
^aTest conditions: Exposure type, temperature range, concentration of solution, test duration or cycles tested.
^bTest type: SLS = single lap shear; DLS = double lap shear.
^cType of FRP: CFRP = carbon FRP; GFRP = glass FRP; BFRP = basalt FRP; Plate = refers to bonding of pultruded or prefabricated laminates; Sheet = refers to bonding by the wet lay-up technique.
^dFailure modes. Refers to conditioned samples only, since control samples typically fail within a thin layer of concrete. C = thin layer concrete failure; A = failure at the adhesive/concrete interface with little or no concrete remaining attached; P = failure at primer interface; NR = not reported; AF = failure at adhesive/FRP interface.

Failure modes listed in Table 1 are consistent with previous work which has found that the failure planes shift from a thin layer within the concrete substrate to the adhesive/concrete interface with little or no concrete left bonded (Shrestha et al. 2015). As concrete strength increases over time, especially under moisture conditions (Wood 1991), and the adhesive deteriorates, the failure plane generally moves away from the concrete substrate.

ELONGATION OF THE EFFECTIVE BOND LENGTH

Characterizing the interfacial bond mechanism as accurate τ-δ relationships has been of much interest to researchers studying the bond behavior of FRP-to-concrete joints, as they provide more insight into the bond than strength alone. It is well known that bonded lengths, \( L \) should be sufficiently long so as to be greater than the effective bond length, \( L_{\text{crit}} \) which ensures that the full range strain development that occurs during testing is captured (Chen and Teng 2001). The concept of the effective bond length is also critical to deteriorated samples since \( L_{\text{crit}} \) generally increases (Mukhopadhyaya et al. 1998; Kabir et al. 2012). Consider the idealized τ-δ relationship in Figure 1b for a control sample, shown as the Undeteriorated curve, which corresponds to the P-δ relationship in Figure 1a. With environmental loading, shown as the Deteriorated curve, the effective bond length, \( L_{\text{crit}} \) increases such that the strength of the bond drops in Figure 1(a) and the peak bond stress, \( \tau_{\text{max}} \) drops in Figure 1(b). In this scenario only a portion of \( P_{\text{IC}} \) will be recorded and \( \delta_{\text{max}} \) cannot be quantified.

![Figure 1 Elongation of the effective bond length with deterioration](image-url)
EXTRACTION OF BOND PROPERTIES

Limitations to Test Data

In order to extract bond properties from individual samples across numerous studies, certain criteria are required:
1. Tests must simulate mode II (in-plane shear) sliding fracture in IC debonding failures;
2. Specimens must have FRP bonded prior to conditioning;
3. The undeteriorated control sample must be free from sustained loading;
4. It is crucial that that the bonded length, \( L \) be greater than \( L_{\text{crit}} \). Effective bond lengths of test specimens in the ambient condition are checked, where possible, against the well-recognised bond strength models and experimental strain readings. Where \( L_{\text{crit}} > L \) the result will be omitted;
5. Double shear tests are simplified to an equivalent single shear tests by considering half of the load and one bonded side (Lu et al. 2005; Wu et al. 2009).

Mechanics Based Approach

A mechanics based approach is used to extract \( \tau - \delta \) characteristics from the available test data (Haskett et al. 2008). Consider an axially loaded FRP laminate externally bonded to a concrete substrate prism in Figure 2. A force in bonded FRP plate \( P_p \), is chosen such that at length \( L \), \( P_p = 0 \) thus a boundary value problem is produced. To solve this boundary value problem an iterative shooting technique is utilized where the value of \( P_p \) is searched so as to satisfy the boundary conditions of \( \delta_1 = \delta_1' = 0 \). The FRP-to-concrete interface bonded length is sliced into \( n \) segments of equal and very short length \( L_s \). A constant axial displacement, force and stress are assumed over each individual segment. Consider segment 1: the plate slips \( \delta \), the strain the plate is then \( \varepsilon_p \), rigidity characteristics of the plate provide \( P_p \), the strain in the concrete prism opposing the plate is \( \varepsilon_c \). A bond force, \( B_1 \) of magnitude \( \tau_1 L \) per \( L_s \) opposes and reduces the plate force \( P_p \), \( \tau_1 \) is a function of the \( \tau - \delta \) relationship, so the plate force at the end of segment becomes \( P_p - B_1 \). The slip strain \( \delta_1' \) is \( \varepsilon_p - \varepsilon_c \), and multiplying \( \delta_1' \) by the segment length, \( L_s \) gives the change in slip over the segment \( \Delta \delta_1' \), thus the slip at the next segment is \( \delta - \Delta \delta_1 \) and the process is repeated through all segments. The slip is then incremented until it is equal to the slip at unloading in \( P - \delta \) relationship. Where the effective bond length has been attained \( L_{\text{crit}} < L \), full interaction boundary conditions are \( \delta = \delta' = 0 \). For laminates where \( L_{\text{crit}} > L \), the boundary conditions is \( \varepsilon_p = 0 \). Material properties are assumed to remain constant, and deterioration is restricted to changes in bond characteristics.

The rigidity characteristics of FRP laminates are found using the rule of mixtures, shown by Seracino et al. (2007) to be highly correlated with experimentally measured plate moduli. Thus, the role of the structural adhesive in determining the equivalent stiffness of the FRP bonded system is accommodated by

\[
E_p = f E_a + (1 - f) E_f
\]

(1)

where \( E_f \) is young’s modulus of the FRP, \( E_a \) is the tensile modulus of the adhesive, \( f \) in Eq. 2 is the thickness fraction, \( t_f \) is the thickness of the FRP and \( t_a \) is the thickness of the adhesive. To apply the rule of mixtures to sheet bonded specimens, a gross laminate thickness of 1.0 mm is assumed. The numerical procedure may then be applied by first idealizing the form of the \( \tau - \delta \) relationship as seen in Figure 3. This is a modified from of the widely accepted CEB-FIB Model Code (CEB 1993) for steel bars embedded in concrete and is chosen for its fit with experimentally the observed \( P - \delta \) relationships.
The function defining the form is

\[
\tau(\delta) = \begin{cases} 
\tau_{\text{max}} \left( \frac{\delta}{\delta_{1}} \right)^{0.4} & \text{for } 0 \leq \delta \leq \delta_{1} \\
\tau_{\text{max}} - \left( \frac{\tau_{\text{max}} - \tau_{f}}{\delta_{\text{max}} - \delta_{1}} \right) (\delta - \delta_{1}) & \text{for } \delta_{1} \leq \delta \leq \delta_{\text{max}} \\
\tau_{f} & \text{for } \delta > \delta_{\text{max}}
\end{cases}
\]

where \(\tau_{\text{max}}\) in Eq. 3 is the maximum bond stress, \(\delta_{1}\) is the slip at \(\tau_{\text{max}}\), \(\delta_{\text{max}}\) is the slip at the initiation of debonding, and \(\tau_{f}\) is the frictional component of bond stress. Then using the numerical procedure, bond properties are varied to generate a \(P-\delta\) relationship corresponding to a particular sample. In Figure 4(a), a \(P-\delta\) curve, specific to a control sample is generated, which allows for the extraction of bond properties in Figure 4(b). Here \(\tau_{\text{max}}\) is found to be 10.0 MPa, \(\delta_{1}\) to be 0.0500, \(\delta_{\text{max}}\) to be 0.120 and \(\tau_{f}\) to be 2.80 MPa. The process is repeated for all 165 samples collected for this study of which 90 tests satisfy the criteria of \(L > L_{\text{crit}}\).

Existing predictive models have been found to be largely inapplicable to the results collected in this study as test variables are outside their bounds. Most commonly, the elastic moduli of the bonding adhesive is exceeded (Chen and Teng 2001), the elastic moduli of the FRP is under (Lu et al. 2005) or over (Wu and Jiang 2013) the bounds of the model, the FRP layer is too thick (Wu and Jiang 2013), it is not applicable to basalt FRP (Chen and Teng 2001; Lu et al. 2005; Seracino et al. 2007; Wu et al. 2009; Wu and Jiang 2013) the model is specific to only cylinder compressive strengths (Seracino et al. 2007; Wu et al. 2009; Wu and Jiang 2013) or the model is not formulated for application to double shear tests (Seracino et al. 2007; Wu and Jiang 2013).

BOND CHARACTERISTIC DETERIORATION FACTORS

A high degree of scatter exists amongst the available test data due to differences in test set-ups, details of environmental conditioning regimes, material properties and geometries. Thus, an accurate model describing the change in bond properties with a certain type of exposure over time or number of cycles is not possible. Rather conservative deterioration factors may be determined through statistical analysis. First extracted bond properties \(\tau_{\text{max}}\), \(\delta_{1}\), \(\delta_{\text{max}}\) and \(L_{\text{crit}}\) are normalized with their corresponding control samples. Then the extent of deterioration between various types of environmental exposure is compared by means of a two sample t-test. It is found that exposure to water, saltwater and acid solutions exhibit similar deterioration and may be combined. Failure at the adhesive/concrete interface with little or no concrete remaining attached is reported in all three conditions. This may indicate that the deterioration is caused by the presence of water irrespective of the solution, or that the concentrations of salt and acid considered may be too low or too high to amount significant changes.
In contrast, all other exposure types provide highly dissimilar levels of deteriorations and must be treated separately. Six potential outliers of 240 data points (considering all values of $\tau_{\text{max}}$, $\delta_1$, $\delta_{\text{max}}$ and $L_{\text{crit}}$) are identified by the Extreme Studentised Deviate (ESD) test (Rosner 1983) and removed. The remaining data is then analyzed to reveal the 5th and 95th percentiles of bond deterioration as summarized in Table 2 along with the bounds to which they apply. A 5th percentile is assigned to $\tau_{\text{max}}$ as it is expected to decrease with exposure, while a 95th percentile is assigned to $\delta_1$, $\delta_{\text{max}}$ and $L_{\text{crit}}$ as these generally increase. Since the proposed factors are derived from a limited number of tests, they are not recommended for use in design, rather to gauge the expected level of deterioration and aid in the design of future shear bond tests.

### Table 2 Bond characteristic deterioration factors

<table>
<thead>
<tr>
<th>Environmental exposure</th>
<th>Normalized bond characteristic</th>
<th>5th Percentile</th>
<th>95th Percentile</th>
<th>Bounds</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aqueous conditions</td>
<td>$\tau_{\text{max}}$</td>
<td>0.359</td>
<td>2.33</td>
<td>2.05</td>
</tr>
<tr>
<td>30% sustained loading</td>
<td>$\delta_1$</td>
<td>0.147</td>
<td>0.93</td>
<td>0.94</td>
</tr>
<tr>
<td>Freeze-thaw cycling for air-entrained concrete</td>
<td>$\delta_{\text{max}}$</td>
<td>0.231</td>
<td>3.78</td>
<td>3.22</td>
</tr>
<tr>
<td>Freeze-thaw cycling for air-entrained concrete and 30% sustained loading</td>
<td>$L_{\text{crit}}$</td>
<td>0.00</td>
<td>43.6</td>
<td>0.86</td>
</tr>
<tr>
<td>Freeze-thaw cycling for normal concrete</td>
<td>$\tau_{\text{max}}$</td>
<td>0.186</td>
<td>3.75</td>
<td>3.22</td>
</tr>
<tr>
<td>Freeze-thaw cycling in 4% NaCl for normal concrete</td>
<td>$\delta_1$</td>
<td>0.161</td>
<td>1.75</td>
<td>1.17</td>
</tr>
<tr>
<td>Sub-zero temperatures</td>
<td>$\delta_{\text{max}}$</td>
<td>0.647</td>
<td>1.95</td>
<td>1.06</td>
</tr>
<tr>
<td>Elevated temperatures</td>
<td>$L_{\text{crit}}$</td>
<td>0.767</td>
<td>2.30</td>
<td>1.53</td>
</tr>
</tbody>
</table>

A graphical indication of the influence of the various test conditions is possible by superimposing degraded $\tau$-$\delta$ relationships using the deterioration factors in Table 2, as illustrated in Figure 5.

![Figure 5 Influence of various environmental conditions on the $\tau$-$\delta$ relationship](image)

The ascending branch is shaped by the chemical adhesion between the FRP, adhesive and concrete, however as micro-cracks develop along the concrete substrate surface, softening is initiated at slip, $\delta_1$ corresponding to the point the maximum shear bond stress, $\tau_{\text{max}}$ is reached, and descends until the initiation of debonding at slip, $\delta_{\text{max}}$ (Yuan et al. 2004). Consider the normalized changes to the $\tau$-$\delta$ relation in Figure 5, the introduction of environmental effects significantly decreases the stiffness of the ascending branch of the $\tau$-$\delta$ relationship. This corresponds to the breakdown of the chemical adhesion provided by the adhesive, damage via micro-cracking of the concrete substrate bonded surface or a combination of both. Post-peak behavior becomes increasingly ductile with exposure to freeze-thaw cycling and aqueous conditions. Post-peak behaviors under varying temperatures remain largely unchanged indicating that material characteristics may remain unchanged.
CONCLUSIONS

A mechanics based approach is used to quantify the change in bond characteristics for degraded FRP-to-concrete joints under a range of environmental conditions. Given the limited reporting of the $P$-$\delta$ relationship and implications of elongating effective bond lengths, an accurate predictive model cannot be formulated from the extracted bond properties. A statistical analysis of the extracted bond properties revealed that exposure to water, saltwater and acid solutions provide similar levels of deterioration and may be combined. Conservative bond characteristic deterioration factors are proposed which allow for the design of future bond tests such that the extension of the effective bond length may be accommodated and the full-range $P$-$\delta$ behaviour be captured.

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ABSTRACT

Flexural strengthening of reinforced concrete (RC) structures by adhesively bonded fibre reinforced polymer (FRP) laminates has become an increasingly popular retrofitting method for RC structures. The effectiveness of such externally bonded FRP strengthening methods depends on the interfacial shear stress transfer mechanism of the bonded interface (Smith and Teng 2002a, b). Therefore, understanding the behaviour of FRP-to-concrete bonded interfaces is of critical importance in determining when the failure will occur and how effectively the FRP is utilized. Many studies have been carried out on understanding and modelling of the FRP-to-concrete bonded interfaces under interfacial shear stresses (Yao et al. 2005; Yuan et al. 2004; Lu et al. 2005).

A bond-slip model, which depicts the relationship between the local interfacial shear stress and the relative slip between the two adherents, is of fundamental importance to understand and model the behaviour of FRP strengthened steel structures. Different bond-slip models with different levels of sophistications have been proposed (Lu et al. 2005). Among which, the bi-linear bond-slip models with a linear ascending branch followed by a linear descending branch are the most widely used owing to its simplicity. Bi-linear bond-slip models are often used to predict the mode-II behaviour in mixed mode cohesive zone models in predicting debonding failures of bonded joints (Teng et al. 2015). In such models, damage evolution is defined using a scalar damage variable which varies from a value of 0 at the initiation of damage to a value of 1 at the full interfacial damage. This damage scalar variable is defined assuming interfacial behaviour is characterized by damaged elasticity and any plastic deformations of the interface are ignored. However, experimentally obtained bond-slip behaviour of FRP-to-concrete bonded joints under cyclic loading showed non-zero residual slip (i.e. slip at zero interfacial shear stress) when local interfacial shear stress is unloaded to zero (Ko and Sato 2007; Mazzotti and Savoia 2009). In addition,
bond-slip behaviour under cyclic loading showed remarkable hysteresis loops, indicating inelastic deformations
occurred during the cyclic loading.

Based on these experimental observations, several analytical bond-slip models have been proposed for FRP-to-
concrete bonded joints under cyclic loading (Ko and Sato 2007; Martinelli and Caggiano 2014; Carrara and De
Lorenzis 2015). Except Ko and Sato (2007) model, the other existing models assumed bi-linear bond-slip
behaviour, as commonly assumed in FRP-to-concrete bond-slip models under quasi-static monotonic loading. The
essential difference between the different bond-slip models proposed for cyclic loading is in the approach used to
define the damage parameter under cyclic loading. In the most advanced of these models (Martinelli and Caggiano
2014; Carrara and De Lorenzis 2015), damaged parameter is either related to the ratio between inelastic and total
fracture energy (Martinelli and Caggiano 2014) or negative slip increments (i.e. reversal of the residual slip at zero
shear stress) (Carrara and De Lorenzis 2015). With the definition of damage parameter in Martinelli and Caggiano
(2014), a negative slip may occur at zero interfacial stress during unloading at higher damage values, thus
resulting in increase of the total energy during the loading unloading process. This is essentially caused by taking
the energy dissipated during degradation of the stiffness (i.e. energy dissipated due to cracking) into account twice
when defining the damage parameter. Carrara and De Lorenzis (2015) model assumed negative shear stresses are
not possible under the assumption that friction and interlocking are negligible. With this assumption, Carrara and
De Lorenzis (2015) assumed that once the interfacial shear stress becomes zero during the unloading process, slip
value will start to reduce while maintaining interfacial shear stress at zero value. However, this assumption does
not agree with the existing experimental results of FRP-to-concrete bonded joints under cyclic loading, where clear
negative values of the shear stresses were observed.

This paper presents the results from a series of simple pull-off test of FRP-to-concrete bonded joints aimed at
investigating the behaviour of FRP-to-concrete bonded joints under cyclic loading. Presented results are a part of
an ongoing study at the University of Queensland (UQ) aimed at understanding and modelling the behaviour of
FRP-to-concrete bonded joints under cyclic loading. Experimental results from six specimens are presented and
discussed against the assumptions made in existing theoretical models for modelling FRP-to-concrete behaviour
under cyclic loading.

MATERIALS AND TESTING METHOD
Specimen Details and Material Characteristics

In total six single lap shear pull off specimens were prepared and tested as a part of an ongoing research work at
UQ structures laboratory. Nominal dimensions of the specimens are given in Figure 1a. The 28-day compressive
strength of concrete, determined by standard concrete cylinder test, was 49.7MPa. The mean value of the concrete
compressive strength tested from cylinders at the time when the pull-off tests were carried out was 64.4 MPa.

Normal modulus CFRP pultruded plates (Sika Carbodur S512) with 50mm width and 1.2mm thickness was
employed in this test. The elastic modulus of the CFRP pultruded pates in the fibre direction was determined from
the test to be 165GPa. A 1.3m long CFRP plate was employed to minimize any effects due to misalignment of the FRP plate and the centre of the actuator.

Sikadur-30 adhesive was used to bond CFRP plate to the concrete substrate. The ultimate tensile strength of the adhesive was determined to be 25.3MPa through adhesive coupon tests. A 50mm gap was provided between the concrete edge at the loaded end and the start of the bond joint, as recommended by Yao et al. (2005) to avoid the edge effects. This gap was prefilled with silicon to better control the gap dimensions. The bonding area (50×300 mm) on the surface of the concrete block was roughened by a needle gun to expose the coarse aggregates (except M1, which was grinded with a hand grinder). Then the dust on the surface was removed using compressed air. The overall test set-up and specimen detail can be seen in Figure 1a. Out of the six identical samples tested, three were tested under quasi-static monotonic loading (named M1, M2 and M3) and three were tested under quasi-static cyclic loading (C1, C2, C3). Before bonding the CFRP plate to the concrete block, voids within the roughened area were filled with the adhesive from the same batch used to bond the CFRP plate. The thickness of the adhesive layer was controlled to be 1mm by placing 1mm thickness aluminium tabs beside the bond area while using a roller to squeeze the excessive adhesive. The bonded specimens were left to cure for at least two weeks before testing.

Testing Procedures and Measurements

The test rig used for single shear pull tests is shown in Figure 1a. In all the specimens, axial strains at the top of the CFRP plate were measure using a digital image correlation (DIC) system. Two LVDTs were employed at the loaded end and the far end to measure any slip between the FRP plate and the concrete. For all samples, two strain gauges were attached to the top and bottom of the CFRP plate within the un-bonded length, 100mm from the loaded end to measure the CFRP plate elastic modulus.

The axial strain of the CFRP plate at any point within the bond length was taken as the mean strain value obtained from five points on the CFRP plate along the transverse direction (width direction). Meanwhile, the interfacial slip and shear stress at different locations were calculated from the strain values using the equations given in Pham and Al-Mahaidi (2007).

For all tests, load was applied using a 100kN capacity MTS servo-hydraulic actuator. Both the quasi-static monotonic tests and the quasi-static cyclic tests were carried out at a displacement rate of 0.05mm/min. In the quasi-static monotonic tests, the samples were loaded till full debonding of the plate. While in the quasi-static cyclic tests, the samples were unloaded to zero force and re-loaded at predefined displacement intervals. Unloading displacement intervals were determined based on the test results from quasi-static monotonic tests to ensure several loading-unloading cycles are achieved within the ascending and descending branches of the local bond-slip curves at different locations along the bond length. The test setup during the test is illustrated in Figure 1b.

![Figure 2 Failure modes of the tested samples](image)

**TEST RESULTS**

**Failure Mode**

The previous study showed that the failure of such kind of bonded joints typically occur within concrete few millimetres away from the bond-line (Yao et al. 2005). Figure 2 shows the failure mode of all the samples. It can be seen that except sample M1, in all other specimens there were a large amount of concrete debris attached to the
debonded plate, indicating these samples failed within the concrete substrate. The main reason for the combined cohesion and adhesion failure occurred in M1 is believed to be the improper surface preparation. It can be seen that with use of needle gun to roughen the surface, combined cohesion adhesion failure could be avoided and failure occurred due to cohesion failure within concrete.

**Load vs Displacement**

Load-displacement curves of the quasi-static monotonic and cyclic tests are shown in Figures 3a and b respectively. In these graphs, load values were taken from the actuator while displacements were taken as the slip at the loaded end calculated from the strain values. The load-carrying capacity of the specimens M1, M2 and M3 were 25.5kN, 28.1kN and 26.6kN respectively. In specimen M1, once the ultimate load is reached, debonding propagation was quite rapid resulting in only few readings within the post ultimate load-displacement curve before the full failure is reached. Specimen M2 showed a short plateau in the load-displacement curve before complete debonding of the interface. However, specimen M3 showed much more gradual failure with a relatively long plateau in the load-displacement curve once the ultimate load is reached.

Compared to the M1-M3 specimens, C1-C3 specimens showed more scatter in the ultimate load readings (Figure 3b). Specimens C1, C2 and C3 resulted in ultimate loads of 26.4kN, 29.9kN and 32.2kN respectively. All C1 to C3 specimens showed a reasonable plateau in the envelope load-displacement curves before complete failure. During the unloading and reloading curves, significant stiffness reduction could be observed due to propagation of debonding. While the curves showed significant scatter, M2 and M3 load-displacement curves agreed well with the envelope load-displacement curves of specimens C2 and C1 respectively (Figures 3c and d).

**Bond Behaviour**

Due to relatively brittle failure of sample M2, data from that specimen didn’t show much debonding propagation, thus omitted in the discussions related to bond behaviour. Strain distributions along the bond length at different displacement values on the load-displacement curves of specimens M1 and M3 (Figure 3a) are shown in Figure 4a. At low displacement values (i.e. displacement I in Figure 3a), strain distribution along the bond length closely matched between the two specimens M1 and M3. This is no surprise considering the close matching of the load-displacement behaviour of the two specimens at lower loads (Figure 3a). As the displacement increased, small differences could be observed in the strain distributions of the two specimens with specimen M3 showing slightly higher strain values for majority of the bond length. At displacement III (i.e. 0.461mm), a relatively constant strain
values could be seen closer to the loaded end indicating debonding (Figure 4a). Considering specimen M3 showed a slightly higher load than M1, strain readings of the debonded region of M3 can be expected to be higher than the strain readings of M1 within the debonded region.

Strain distributions along the bond length of the specimens M3 and C1 at different load levels (Figure 3d) are compared in Figure 4b. While the load is only 20kN (point A), strain distributions of both specimens agreed well. As the load increased to 23.5kN (point B), strain values of C1 closer to the loaded end found to be lower than those of M3. At load 25.7 (point C), strain distributions of the two specimens showed more pronounced differences. From the strain values at the loaded end of C1 and M3 its clear that the debonding of C1 initiated at a lower strain value than in M3. At load 25.7kN (point C), strain values of C1 closer to the loaded end found to be constant, thus indicating debonding within this region. However, strain values of M3 closer to the loaded end still showed some variations, thus showing interfacial shear stress transfer. Nevertheless, about 50mm away from the loaded end, strain values of specimen M3 becomes relatively constant, thus indicating debonding.

![Figure 4 The strain distribution along the bonding length a) Strain distribution of monotonically loaded samples at different stages; b) Comparison between sample M3 and C1 at different load levels.](image)

The interfacial shear stress distributions along the bonding length at different loading points are illustrated in Figure 5 for specimens C1, C2 and C3. Corresponding unloading and reloading points are indicated in Figure 3b for each specimen. In specimens, C1 and C2, with the increase in displacement, peak shear stress moves away from the loaded end. This indicates damage propagation along the bond length. In specimen C3, drop of the shear stresses to zero at the loaded end with the increase in load can be observed, which indicates the initiation of damage.

![Figure 5 Bond shear stress distributions of specimens C1, C2 and C3 at different loading points](image)

The bond-slip curves at different locations along the bond length of specimens M1, M2 and M3 are shown in Figures 6a, b and c respectively. Due to the brittle failure of the specimen M2, complete bond-slip curves could not be obtained for this specimen. Nevertheless, peak shear stress at several locations along the bond length could be observed, thus the curves are shown in Figure 6b. In all the specimens, peak bond shear stress showed higher values closer to the loaded end (approx. 10MPa in specimens M1 and M3 and 7MPa in specimen M2), and gradually reduced along the bond length. In specimens M1 and M3, peak bond shear stress converged to a value of approximately 5MPa after about 45mm length away from the loaded end.

The bond-slip curves of the Specimen C1 at different locations are shown in Figures 6d-f, while those for specimens C2 and C3 are shown in Figures 6g-i and 6j-l respectively. Similar to specimens M1-M3, peak bond shear stress of specimens C1-C3 also showed higher values on locations closer to the loaded end compared
those from locations away from the loaded end. Within the softening region of the curves, unloading showed a residual slip at zero bond shear stress. This residual slip tends to increase with the slip at the unloading point. In addition, bond shear stresses tend to be negative with further unloading beyond zero shear stress point. This is different to the assumption made in Carra and De Lorenzis (2015).

Figure 6 Bond-slip relations for different samples: a) M1; b) M2; c) M3; d) C1 at 13mm from the loaded end; e) C1 at 28mm from the loaded end; f) C1 at 43mm from the loaded end; g) C2 at 13mm from the loaded end; h) C2 at 28mm from the loaded end; i) C2 at 43mm from the loaded end; j) C3 at 13mm from the loaded end; k) C3 at 28mm from the loaded end; l) C3 at 43mm from the loaded end; m-o) Comparison of the bond slip relation between M2 and C1 at m) approx. 39mm, n) approx. 48mm, o) approx. 59mm.
A comparison of the bond-slip curves from specimens M3 and C1 are given in Figures 6m-o. It can be seen that closer to the loaded end, envelope bond-slip curve of the C1 specimen matched closely with the bond-slip curve of M3 at the same location (Figure 6m). However, further away from the loaded end bond-slip curves from the two specimens showed some discrepancy towards latter part of the softening branch of the curves (Figures 6n and o). Towards the latter part of the softening branch of the bond-slip curves, bond-stress of the curves from specimen C1 was higher than the bond-stress observed from specimen M3. This indicates that within this region of the bond length, interfacial stress transfer still occurs in specimen C1 while complete debonding has occurred in specimen M3. The reason behind this higher stress transfer observed in specimen C1 could be due to mechanisms such as friction and interlocking. These mechanisms vary from one location to another and between specimens, which is observed in the differences of bond-slip curves at different locations, especially within the latter part of the softening branch of the bond-slip curves.

CONCLUSIONS

This paper presents the results from a series of simple pull-off tests of FRP-to-concrete bonded joints under quasi-static monotonic and cyclic loading. The results of these tests are expected to provide a better understanding of the behaviour of FRP-to-concrete bonded joints subjected to cyclic loading. Load-displacement behaviour, strain distribution along the bond length at different loads, stress-distributions along the bond length at different loads, and bond-slip curves at different locations along the bond length are presented and discussed. Load-displacement curves showed obvious stiffness degradation as the curve becomes nonlinear. Stiffness degradation was also seen in the bond-slip curves when unloaded-reloaded within the softening region of the bond-slip curve. During the unloading from softening region of the bond-slip curves, a clear residual slip was seen at zero bond shear stress. Therefore, the damaged elasticity assumption made in some of the existing theoretical models for bond-slip behaviour under cyclic loading found to be inaccurate. Bond-shear stress became negative with further unloading. This indicates that reduction of the slip at constant zero shear stress assumption made in some of the existing theoretical models is inaccurate. Bond shear stress values towards the latter portion of the softening part of the bond-slip curves tends to vary from one location to another within the same specimen, and between the specimens indicating possible effects due to friction and interlocking mechanisms within the cracked regions.

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STRUCTURAL RESPONSE OF FRP-TO-CONCRETE JOINTS UNDER CYCLIC ACTIONS: FRACTURE MECHANICS AND FATIGUE ANALYSIS

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ABSTRACT

Fiber-Reinforced Polymers (FRPs) are getting more and more common in structural strengthening of Reinforced Concrete (RC) members, also in seismic areas. However, the current knowledge about their mechanical behaviour and interaction with the existing materials is based on the big deal of research carried out in the last twenty years with almost exclusive reference to the case of monotonic loads. Therefore, in the last years both experimental tests and theoretical studies have been presented in the scientific literature for scrutinising the response under cyclic actions of FRP strips glued to concrete. Based on a model recently proposed by the Authors, this paper aims at unveiling the relationship between fracture-mechanics-related parameters and the resulting fatigue response of FRP-to-concrete adhesive joints. Moreover, it shows the influence of physical quantities and mechanical properties on the resulting fatigue curves.

KEYWORDS

FRP, concrete, strengthening, cyclic actions.

INTRODUCTION

Although, nowadays, the use of Fiber-Reinforced Polymer (FRP) materials is fairly common as a technical solution for strengthening and retrofitting existing constructions (Teng et al. 2001), the current codes and guidelines (ACI 2008; CNR 2013) do not provide practitioners with specific rules for the design under cyclic actions, which is obviously of relevance in seismic areas. As a matter of fact, plenty of experimental (Chajes et al. 1996) and theoretical (Cornetti and Carpinteri 2011) studies have been carried out for investigating the cracking processes leading to debonding of FRP strips Externally Bonded (EB). Conversely, the researches addressing the behaviour of FRP and, specifically, the possible debonding failure, under cyclic loads are much more recent (Carloni et al. 2012) and are mainly experimental in nature.

As for modelling, an empirical bond-slip model intended to simulating the behaviour observed in a series of monotonic and cyclic tests, carried out on Aramid (A), Carbon (C) and Polyacetal (P) FRP strips glued on concrete blocks, can be found in the literature (Ko and Sato 2007). The model is based on assuming a Popovics-like bond-slip law and seven mechanical parameters need to be calibrated experimentally, as a result of the aforementioned empirical nature of the model under consideration.

More recently, Carrara and De Lorenzis (2014) proposed a coupled damage-plasticity model intended at simulating the cyclic behaviour of FRP-to-concrete interfaces under cyclic loads, based on a classical bilinear elastic-softening bond-slip law.

In the same period, the Authors followed an alternative approach, based on Fracture Mechanics concepts, for formulating a model capable to simulating the cyclic behaviour of FRP strips externally bonded to concrete (Martinelli and Caggiano 2014). Two alternative bond-slip laws (namely, linear-exponential and doubly exponential, along with the aforementioned bi-linear one) were originally considered: one of the most attractive features of the proposed model is that the main physical quantities of relevance in Fracture Mechanics, such as the work spent in the fracture process, are derived in closed-form.

This paper is intended as a further contribution to modelling FRP-to-concrete adhesive joints under cyclic actions. Based on a model recently proposed by the Authors, this paper aims at unveiling the relationship between fracture-mechanics-related parameters and the resulting fatigue response of FRP-to-concrete adhesive joints. Moreover, it
shows the influence of physical quantities (such as the bond length) and mechanical properties (such as fracture energy) on the resulting fatigue curves.

Section 2 outlines the key theoretical assumptions of the aforementioned model, whereas Section 3 proposes a parametric analysis and shows the fatigue curves. Finally, the main findings of this study are highlighted in Section 4, along with the future developments of this research.

OVERVIEW OF THE ANALYSIS METHOD

The Authors have recently formulated a numerical model capable of simulating the cyclic response of FRP-to-concrete joints (Martinelli and Caggiano 2014). It is based on the following fundamental assumptions:

− the cracking process develops at the FRP-to-concrete interface in pure “mode II”;
− the softening branch of the bond-slip law is assumed “a priori” and takes an exponential expression;
− stiffness degradation and damage evolution in the unloading stages depend upon the current value of the “fracture work” spent at each point of the FRP-to-concrete interface;
− “small” displacements are assumed at the interface and the concrete substrate is assumed to be rigid.

Although, in its original formulation, the model features two alternative expressions for the bond-slip law, the present study is based upon assuming the following linear-exponential relationship:

\[
\begin{align*}
\tau(z) &= -k_E s(z) & \text{if } s(z) \leq s_e \\
\tau(z) &= -\tau_0 e^{-\beta [s(z) - s_e]} & \text{if } s(z) > s_e
\end{align*}
\]

(1)

where \( s(z) \) and \( \tau(z) \) are, respectively, interface slip and shear-bond stress at the considered \( z \) abscissa, \( k_E \) is the tangential bond stiffness defining the pre-peak branch bond-slip law, \( s_e = \tau_0/k_E \) represents the elastic slip value, \( \tau_0 \) is the corresponding shear strength, while \( \beta \) is a parameter of the post-peak softening branch of the \( \tau \)-s law. Figure 2 depicts the shape of this interface bond-slip law and highlights its relevant mechanical parameters.

Under the aforementioned assumptions, the fracture energy \( G_f^II \) (hereafter expressed by \( G_F \), for the sake of simplicity) can be easily expressed as a function of the three independent parameters \( k_E \), \( s_e \) and \( \beta \).
Therefore, the damage parameter \( d \) can be defined in each point of the adhesive interface as follows:

\[
d = \xi^\alpha_d, \quad \text{with} \quad \xi = \frac{\sigma_s}{G^H_f},
\]

where \( \alpha_d \) controls the loading/unloading stiffness \( k \) (Figure 2).

**ANALYSIS**

Martinelli and Caggiano (2014) validated their model by comparing some of the experimental results obtained by Ko and Sato (2007) on both monotonic and cyclic pull-out tests. The present study moves from the parameters employed in that validation (namely, \( L=300 \text{ mm} \), \( E_p=10.4 \text{ kN/mm} \), \( b_p=50 \text{ mm} \), \( k_s=52.22 \text{ MPa/mm} \), \( \tau_0=2.256 \text{ MPa} \) and \( G_f=G_{F,ref}=0.958 \text{ N/mm} \)). The following analyses aim at investigating the influence of cyclic loading protocol, interface properties (Figure 2) and geometry (Figure 1) on the resulting cyclic response. As for the protocol, it is described by a uniform amplitude \( 2\Delta F \) around a mean value \( F_m \) equal to one half of the maximum load \( F_{mon} \), obtained in the monotonic loading process and determined as follows:

\[
F_m = F_{mon}/2 = \sqrt{2G_F E_p p_p b_p}/2.
\]

Since low-cycle fatigue is mainly considered, in this study \( \Delta F \) ranges between \( 0.30 F_{mon} \) and \( 0.45 F_{mon} \). Figure 3 shows the applied-force-maximum-slip response obtained under the four values considered as the load amplitude \( \Delta F \).

Figure 3c shows that, for \( \Delta F = 0.40 F_{mon} \), structural damage develops slowly, as a significant number of cycles is
needed for the FRP strips to be completely debonded from the concrete substrate. Conversely, debonding develops in a much faster way as the cycle amplitude grows up, and few cycle reversals are sufficient for failure to occur. Moreover, although a linear unloading/reloading process is assumed at the local level (Figure 2), a hysteretic response emerges for sufficiently wide force oscillations: it is already apparent under $\Delta F = 0.35 F_{\text{mon}}$ (Figure 3b) and it gets more and more pronounced for $\Delta F = 0.40 F_{\text{mon}}$ (Figure 3c) and $\Delta F = 0.45 F_{\text{mon}}$ (Figure 3d). The relationship between force amplitude $\Delta F$ and the corresponding number of cycle reversals, after which the system fails in debonding, may be regarded in the light of the Fatigue Theory (Suresh 1998). More specifically, the well-known Wöhler curve (also known as S-N curve) can be drawn by relating these two quantities. Particularly, Figure 4 shows the S-N curve obtained for the system under consideration, along with those determined for shorter bond lengths (namely, equal to 100 mm and 200 mm).

![Figure 4 Fatigue curves S-N: the influence of bond length L.](image)

Figure 4 shows the difference in terms of cyclic response for the three considered bond lengths under a load amplitude $\Delta F = 0.40 F_{\text{mon}}$. More specifically, the two graphs highlight that the system is capable of only two cycle reversals for $L = 100 \text{ mm}$ (Figure 5a) and twelve for $L = 200 \text{ mm}$ (Figure 5b), whereas the number of reversals were twenty for $L = 300 \text{ mm}$.

![Figure 5 Cyclic load-slip curves ($\Delta F = 0.40 F_{\text{mon}}$): the influence of bond length L.](image)

Therefore, two remarkable observations have to be drawn out of Figure 4 and Figure 5:

- as was not obvious in principle, the three curves are represented by a straight segment in a log-log plane, and, hence, they can be represented by the following exponential relationship (Suresh, 1998):

$$\frac{2\Delta F}{F_{\text{mon}}} = a \cdot (2N)^b,$$

where the constants $a$ and $b$ depend on both the interface bond-slip law and the bond length; moreover, in principle, they are also influenced by the average stress/force assumed in the cyclic load protocol;

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- as expected, for a given amplitude $\Delta F$, debonding occurs earlier in FRP strips with shorter bond lengths. Furthermore, a series of analyses are carried out by considering a variable value of $G_F$, ranging from the aforementioned $G_F = G_{F,ref}=0.958 \text{ N/mm}$ down to $0.70 \ G_{F,ref}=0.671 \text{ N/mm}$.

Figure 6 reports the cyclic response obtained for the lowest value of $G_F$ considered in this parametric analysis: the four graphs correspond to the ones reported in Figure 3 for the case of FRP-to-concrete joint with $G_F = G_{F,ref}$. More specifically, the graphs in Figure 6 compare the cyclic responses obtained in the two cases of $G_F=0.958 \text{ N/mm}$ and $G_F=0.671 \text{ N/mm}$.

![Figure 6 Cyclic load-slip curves ($G_F=0.70G_{F,ref}=0.671 \text{ N/mm}$).](image)

Besides the two extreme cases reported in Figure 6, Figure 7 shows the S-N curves corresponding to the four values of $G_F$ included between $G_{F,ref}$ and $0.70 \ G_{F,ref}$.

![Figure 7 Fatigue curves S-N: the influence of fracture energy $G_F$ (for a given bond length, $L=300 \text{ mm}$).](image)

It is worth highlighting that, for each $G_F$, the corresponding value of $F_m$ was determined afresh according to eq.(4)
and, hence, the variable values of $F_{mon}$ (and $\Delta F$) have been updated for each $G_F$ (Figure 6). Conversely, the bond length $L$ is kept unchanged; hence, the cases analysed by assuming lower values of $G_F$ result in being characterised by a higher ratio between the actual bond length $L$ and the effective transfer length (Teng et al. 2001). This is the reason why, as shown in Figure 7, the S-N curves determined for lowest $G_F$ is above the ones with higher values of fracture energy.

Moreover, a progressive transition is observed in terms of S-N curves (Figure 7) from the highest to the lowest values of $G_F$, and, however, all points belonging to each series line up on a straight segment, confirming the general validity of a linear relationship in the log-log plane described by eq. (5).

CONCLUSIONS

This paper aims at understanding the debonding of FRP strips glued to concrete and subjected to cyclic loads. Based on the results achieved in this study, the following aspects can be remarked:

- as expected, force amplitude significantly affects the resulting cyclic response of the analysed systems;
- low-amplitude cycles result in high number of reversals, whereas a progressive reduction in reversals (with significant hysteresis of cycles) is observed for wider cycle amplitudes;
- it is unveiled that the number of cycle reversals can be correlated to the corresponding amplitude by means of a linear curve in the log-log plane, as commonly occurs in the Theory of Fatigue;
- more specifically, the results reported in this paper deals with the so-called low-cycle fatigue and shows that the bond length significantly influences position and slope of the resulting curve;
- on the one hand, if the fracture energy is kept constant (and, hence, both $F_{mon}$ and the effective transfer length of the system keeps unchanged), reducing the bond length results in a significant reduction of the number of cycles leading to a complete debonding (namely, a reduction in fatigue life) of the FRP-to-concrete joint;
- on the other hand, if $G_F$ varies without changing the bond length $L$, reducing $G_F$ reverberates on both $F_{mon}$ and the effective transfer length and, hence, the fatigue life obtained in cases of lower fracture energy is higher than the ones with higher $G_F$.

In the Authors’ best knowledge, these results have not been highlighted so far in the international scientific literature. However, this work is intended as a first step towards connecting fracture mechanics and fatigue theory for predicting the cyclic response of FRP-to-concrete joints subjected to cyclic actions.

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REFERENCES


DEBONDING DETECTION OF FRP STRENGTHENED CONCRETE BEAMS BASED ON IMPEDANCE ANALYSIS

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ABSTRACT

For a fiber reinforced polymer (FRP) strengthened concrete structure, minor defects and flaws, such as interfacial debonding, can lead to catastrophic failure. Electromechanical impedance (EMI) technique is proved to be efficient to identify structural damage by using piezoceramic transducers (PZT) bonded on the structure as actuator-sensor. Therefore, with the purpose of SHM, this study presents a simplified spectral model to obtain the impedance signatures of PZT which is bonded on a FRP laminated concrete beam. One of the advantages of the proposed model, which is established in frequency domain, is its efficiency to capture dynamic response in high frequency range. Furthermore, in order to validate the accuracy of proposed model, experimental test was carried out to measure impedance signatures and the results show the capability of presented spectral approach and EMI technique for monitoring debonding damage in FRP-strengthened structures.

KEYWORDS

Electromechanical impedance, FRP, interfacial debonding, damage detection.

INTRODUCTION

Although FRP has attracted great attentions in civil engineering field as an effective way of retrofitting and strengthening due to its high strength to weight ratio and corrosion resistance, structural damage often appears due to the various kinds of loading, temperature changing and severe environment exposure. The premature interfacial debonding is an important structural damage which always leads to a brittle and sudden failure, and it undoubtedly limits the extension of this technique (Teng et al. 2002; Bank 2006). This kind of failure is always initiated at a major flexural crack and then propagates towards the end of composite plate (Pesic et al. 2003), thus, the bond health condition of FRP-strengthened structures should be paid more attentions. However, debonding damage appears as minor defect in its early stage which is very difficult to be detected by using traditional methodologies such as dynamic analysis, for that reason, reliable SHM approach that can identify incipient debonding damage is required.

More recently, piezoelectric sensor is very popular in SHM which can evaluate structural damage based on the measured electromechanical impedance. By bonding PZT patch on the structure as both actuator and sensor, the measurement of impedance is directly related to the mechanical impedance of host structure, and the changes in structural properties can be reflected by the alternation of measurement results in high frequency domain. Up to now, the EMI method has been implemented in numerous applications to identify structural damage. Experimental test were carried out (Park et al. 2005; Min et al. 2012) to monitor the healthy condition of alloy beam, pipe system and spot-welded joints, and the interfacial debonding of FRP-strengthened concrete beam were evaluated (Xu et al. 2010; Park et al. 2011) by using EMI analysis. The results show the capability of PZT sensors for monitoring structural damages, especially minor damages.

Furthermore, numerical simulation has also drawn researchers’ attention in this field. Taking into account that the mesh size of finite element model should be comparable to the wavelength of highest frequency sweep range, it results in huge number of elements and high computational cost (Hamzeloo et al. 2012). In this work, a one-dimensional spectral model is employed to calculate the impedance signatures for bonded PZT-FRP beam, and very limited elements are implemented in the proposed model. Experimental test were carried out to validate the proposed model, it’s clear that this approach might be a potential tool for the identification of FRP debonding damage.
SPECTRAL ELEMENT METHOD

For FRP reinforced concrete segment, spectral model (Doyle 1997; Gopalakrishnan et al. 2008) was established to investigate the mechanical behavior. To this end, Hamilton’s principle was applied to obtain the governing equations, considering the kinematic motions of a two-nodes element with four degree of freedom $u_0$, $w$, $\phi$ and $s$ are shown in Fig 1, which denote the axial displacement, the transverse displacement, the rotation of the beam about y-axis, and the interfacial slip between FRP and concrete, respectively.

![Figure 1 Kinematic motions of FRP-RC beam with PZT patch](image)

The governing equations of the reinforced beam are given as

\[ \delta u_0 = I_0 \ddot{u}_0 - I_1 \dot{\phi} + I_{0\text{FRP}} \ddot{s} - A_1 u_{0xx} + B_1 \dot{s}_{xx} - A_{\text{FRP-xx}} s_{xx} = 0 \]  \hspace{1cm} (1)

\[ \delta w = I_0 \ddot{w} - A_{22} w_{xx} + A_{22} \phi_s = 0 \]  \hspace{1cm} (2)

\[ \delta \phi = I_2 \ddot{\phi} - I_1 \dot{u}_0 - I_{1\text{FRP}} \ddot{s} + B_1 \dot{u}_{0xx} - A_{22} \phi_{xx} - A_{22} \phi + B_{\text{FRP}} s_{xx} = 0 \]  \hspace{1cm} (3)

\[ \delta s = I_{0\text{FRP}} \ddot{s} + I_{0\text{FRP}} \dot{u}_0 - I_{1\text{FRP}} \ddot{s} - A_{\text{FRP-xx}} s_{xx} = 0 \]  \hspace{1cm} (4)

where $\dot{}$ represents temporal derivative and $(\ )_{xx}$ denotes double differentiation with respect to x. The coefficients $A_1$, $B_1$, $D_1$, $A_{22}$, $A_{\text{FRP}}$, $B_{\text{FRP}}$ are related to materials properties and $I_0$, $I_1$, $I_2$, $I_{0\text{FRP}}$, $I_{1\text{FRP}}$ are associated with inertial terms.

Applying Discrete Fourier Transform (DFT), the displacement field introduced previously $\{u\} = \{u_0(x,t), w(x,t), \phi(x,t), s(x,t)\}$ takes the following formulation in frequency domain

\[ \{u\} = \sum_{n=1}^{N} \{ \hat{u}(x, \omega_n) \} e^{-j \omega_n t} = \sum_{m=1}^{M} \left( \sum_{n=1}^{N} \{ \hat{u}^*(n) \} e^{-j \omega_n x} \right) e^{-j \omega_n t} \]  \hspace{1cm} (5)

where $\omega_n$ denotes the n-th circular frequency, N is the number of frequency points and $\hat{u}(x, \omega_n)$ represents the spectral amplitude vector corresponding to the generic displacement vector as a function of $(x, \omega_n)$. A Fourier expansion of $\hat{u}(x, \omega_n)$ has been also carried out in the longitudinal direction where $\{ \hat{u}^*(n) \} \omega_n = \{ \hat{u}_0, \hat{w}, \hat{\phi}, \hat{s} \}$ represents the wave coefficient vector associated with the m-th mode of wave and for each frequency and $k_{mn}$ denotes the m-th wave number related to n-th frequency $\omega_n$. Therefore, the governing formations can be represented as the following polynomial eigenvalue problem:

\[ [W(k)] \{ \hat{u}^* \} = 0 \]  \hspace{1cm} (6)

where $W(k)$ is the matrix polynomial depending on the wavenumbers $k_{mn}$ for each frequency $\omega_n$. In this case, the order of the matrix polynomial is 8. Thus, there are eight eigenvalues ($k_{mn}$ where $m = 1, \ldots, 8$) and eigenvectors ($\{ \hat{u}^* \}$). After solving the eigenvalue problem and combing boundary force conditions, the expression of spectral element model is obtained as follows

\[ \begin{bmatrix} f_1 \\ f_2 \end{bmatrix} = [T_2][A][T_2]^{-1} \begin{bmatrix} \{ \hat{u}_1 \} \\ \{ \hat{u}_2 \} \end{bmatrix} \begin{bmatrix} \{ \hat{u}_1 \} \\ \{ \hat{u}_2 \} \end{bmatrix} \]  \hspace{1cm} (7)
where \([T_1]\) and \([T_2]\) are obtained from governing equations and boundary forces respectively, and finally the dynamic stiffness matrix \([K]\) for the spectral PZT-bonded FRP strengthened concrete element is formed. From the expressions of proposed spectral element, it can be noted that only one element is needed as long as the structure remains uniform in material and geometrical properties which is able to reduce computational cost significantly.

Based on the presented spectral model, the impedance signature of PZT sensor can be evaluated by using formula as follows (Liang et al. 1994)

\[
Y(\omega) = j\omega b_{PZT} \frac{e_{PZT}}{e_{PZT}} (\varepsilon_3^T Y_{31} - \frac{Z_s(\omega)}{Z_s(\omega) + Z_a(\omega)} d_{33}^T Y_{11})
\]

where \(Y(\omega)\) is the admittance (inverse of impedance), \(\omega\) is the angular frequency of the driving voltage, \(d_{33}\) is the piezoelectric strain constant between \(z\) and \(x\) directions at zero stress, \(j\) is the imaginary unit. Furthermore, \(Z_s(\omega)\) and \(Z_a(\omega)\) are the mechanical impedances of the PZT and the host structure, they are able to be evaluated easily as long as the displacement field are obtained in frequency domain by applying proposed spectral approach.

RESULTS

With the purpose of validating the feasibility of the presented model, experimental tests were performed on a concrete block strengthened with FRP plate. The dimensions of the specimen are \(31.3 \times 9.5 \times 7.5 \text{cm}^3\) and the dimensions of the FRP reinforcement are \(29 \times 5 \times 0.12 \text{cm}^3\). As shown in Fig 2, PZT sensors were bonded to the external face of the FRP reinforcement using an epoxy adhesive distributed along the surface of the FRP plate. The three sensors were individually connected to three different channels of a multiplexor, which was used to make a multiple connection between the impedance analyzer and the sensors. The Impedance Analyzer excited the PZT sensors and simultaneously recorded the impedance signatures received by the PZT sensors.

![Figure 2 Experimental set-up](image)

In this work, the output impedance of PZT2 in the middle is selected to compare with numerical results. As presented previously, only one spectral element is enough as long as no discontinuity happens in the structure, thus, as shown in Fig 3 the specimen is represented by 5 spectral elements (E1-E5) along the whole beam with the length of 0.01, 0.14, 0.01, 0.14 and 0.013 respectively. It’s clear that the number of elements is very limited leading to low computational costs compared with conventional finite element model.

![Figure 3 Spectral element mesh distribution](image)
Fig 4 illustrate the comparison of EMI spectra obtained by experimental test and numerical simulation in the frequency range of 10-100kHz. It can be observed that the proposed model is able to predict output impedance with satisfied accuracy considering only five elements were applied.

Figure 4 Comparison of experimental and numerical impedance signatures

To perform the proposed model for damage detection, the interfacial shear modulus of element 3 (E3) is reduced to simulate the debonding damage in the early stage. Root mean square deviation (RMSD) is chosen as the damage indicator in order to measure damage severity quantitatively as follows:

\[
\text{RMSD} = \left( \frac{100}{n} \sum_{i=1}^{n} \left| Z_i(a_0) - Z_i(a_f) \right|^2 \right)^{1/2}
\]

where \( Z_i(a_0) \) is the impedance of the PZT measured at a previous stage, which might agree with the healthy condition of the structure, and \( Z_i(a_f) \) is the corresponding value at a subsequent stage, which might agree with a post-damage stage, at the \( i \)th frequency point; \( n \) is the number of frequency points. In this work, the appearance of debonding in FRP reinforced beam would lead to variation of output impedance signatures in high frequency range particularly, and this incipient damage can be evaluated by using RMSD as damage indicator. It is obvious that RMSD is directly associated with the appearance of damage and increases with the developing of damage severity from 30% to 70%. Therefore, the EMI method is proved to be sensitive enough to identify structural damage such as FRP debonding in this case.

Figure 5 RMSD in different damage level
CONCLUSIONS

A one-dimensional spectral element method was presented in this work. The proposed method was established in frequency domain by using DFT and is able to capture EMI signature in high frequency range. Although very limited elements were employed, its precise is validated by using experimental results. Moreover, numerical simulation is performed to prove that the minor debonding damage of FRP strengthened concrete beam can be reflected according to the variations of the impedance output by introducing RMSD as damage indicators.

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ABSTRACT

Bond behaviour plays an important role in reinforced concrete structures design and performance. In this study, bond performance of glass fibre reinforced polymer (GFRP) reinforced Geopolymer Cement (GPC) concrete and steel reinforced GPC concrete are analysed and compared. The bond failure mode, the average bond strength and the free end bond-slip curves are used for the comparison. Furthermore, Tepfers’ concrete ring model is used to further analyse the splitting failure in ribbed steel and GFRP bar specimens. The results showed that failure mode of the specimen plays a significant role on the average bond stress at failure. Pull-out failure resulted in higher bond strength for ribbed steel bars whereas splitting failure caused higher bond stress for GFRP bars, showing the difference in the bond mechanism between the two bars. For the specimens with splitting failure mode, the ribbed steel bar specimens displayed a higher bond-angle than the GFRP bar specimens explaining the ribbed steel bars higher splitting tendency.

KEYWORDS

Geopolymer concrete, GFRP, bond-slip curves, bond-angles, splitting failure, pullout failure.

INTRODUCTION

Bond behaviour is an important part of both design and performance of reinforced concrete structures. Bond allows forces to be transferred from the reinforcement to the surrounding concrete. Thus the bond behaviour between constituent elements of a structure determines its performance as a whole. Due to the good bond between ribbed steel bars and concrete, steel reinforced concrete is one of the most successful construction materials. However, due to corrosion of steel bars and environmental issues associated with cement production, engineers are looking for alternative materials. Glass fibre reinforced polymers (GFRP) reinforced geopolymer cement (GPC) concrete stand out to be an ideal solution. GFRP bars are manufactured from thermoset polymers vinyl ester and glass fibres and are characterised by high tensile strength, high durability, light weight, and electromagnetic permeability (Bank 2006). Geopolymer cement binder is produced by activating by-product materials that are rich in silica and alumina, such as fly ash and rice husk ash with alkali liquids such as metal hydroxide and/or alkali silicate. In addition to environmental benefits, GPC also provide a rapid rate of strength development, resistance to sulphate attack, acid resistance, little drying shrinkage, low creep, improved resistance to fire, and prolonged handling time (Hardijito and Rangan 2005, Junaid et al. 2014, 2015a, 2015b). Various studies have been carried out on steel reinforced OPC concrete, steel reinforced GPC concrete, and GFRP reinforced OPC concrete (Tepfers 1979, Malvar 1992, Cui and Kayali 2013, Sofi et al. 2007). Based on these studies different standards have been developed to assist in the design of reinforced concrete structures such as ACI 408R-03. However, the knowledge on the bond performance of GFRP reinforced GPC concrete and their comparison with steel reinforced GPC concrete is limited, thus the need for this research.

EXPERIMENTAL RESULTS

The analysis of this paper is based mainly on the experimental results from Tekle et al. (2016) for GFRP reinforced GPC concrete and Cui and Kayali (2013) for steel reinforced GPC concrete. Both studies and the experimental program used pullout specimens were the reinforcement was embedded in 100 × 170 mm concrete cylinder.
Straight non-deformed sand-coated GFRP bars with a nominal diameter of 15.9 mm and elastic modulus of 62.6 GPa were used (Figure 1). These were manufactured using a pultrusion process, and made up of continuous E-glass fibres, with a minimum volume of 65 %, bound together by a modified vinyl ester with a maximum volume of 35 %. Normal ductility hot-rolled ribbed steel bars with 16 mm diameter and with a yield and ultimate strength values of 546 MPa and 633 MPa respectively were used. Plain steel bars with 16 mm diameter and with yield and ultimate strength values of 339 MPa and 507 MPa respectively were also used.

![Figure 1 Reinforcement bars: (a) plain steel bar; (b) sand-coated GFRP bar (c) ribbed steel bar](image)

The GPC concrete used in both experimental programs is fly ash based. The mix proportions and properties of the concrete are as given in Table 1. The pullout test setup used by Cui and Kayali (2013) and Tekle et al. (2016) is as shown in Figure 2.

<table>
<thead>
<tr>
<th>Ingredient / Property</th>
<th>GPC concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (kg/m³)</td>
<td>-</td>
</tr>
<tr>
<td>Fly Ash (kg/m³)</td>
<td>420</td>
</tr>
<tr>
<td>Coarse aggregate (kg/m³)</td>
<td>1090</td>
</tr>
<tr>
<td>Fine Aggregate (kg/m³)</td>
<td>630</td>
</tr>
<tr>
<td>12M NaOH solution (kg/m³)</td>
<td>60</td>
</tr>
<tr>
<td>Na₂SiO₃ solution (kg/m³)</td>
<td>150</td>
</tr>
<tr>
<td>Water (kg/m³)</td>
<td>31</td>
</tr>
<tr>
<td>Superplasticiser (kg/m³)</td>
<td>4</td>
</tr>
<tr>
<td>Viscosity modifier (kg/m³)</td>
<td>4</td>
</tr>
<tr>
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<tr>
<td>Splitting tensile strength (MPa)</td>
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</tr>
</tbody>
</table>

The experimental results are summarized in Tables 2 and 3 for GFRP and steel bars, respectively. The bond stress is assumed to be uniform along the bar and is defined as the shear force per unit surface area of the bar. This definition of average bond stress is followed throughout the analysis, and is calculated as

$$\tau = \frac{P_{\text{max}}}{\pi d l_b}$$

where $\tau =$ average bond stress, in megapascals; $P_{\text{max}} =$ applied maximum pullout load, in Newtons; $d =$ nominal diameter of the bar, in millimeters; and $l_b =$ bonded/embedded length, in millimeters.
Figure 2 Pullout experiment setup for steel bar

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Embedment length (mm)</th>
<th>Load $P_{\text{max}}$ (kN)</th>
<th>Average bond stress $\tau$ (MPa)</th>
<th>Free end slip (micron)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>G48-1</td>
<td>48</td>
<td>45.8</td>
<td>19.0</td>
<td>242</td>
<td>P</td>
</tr>
<tr>
<td>G48-2</td>
<td>48</td>
<td>43.3</td>
<td>18.0</td>
<td>374</td>
<td>P</td>
</tr>
<tr>
<td>G48-3</td>
<td>48</td>
<td>47.2</td>
<td>19.6</td>
<td>355</td>
<td>P</td>
</tr>
<tr>
<td>G96-1</td>
<td>96</td>
<td>89.1</td>
<td>18.5</td>
<td>289</td>
<td>S</td>
</tr>
<tr>
<td>G96-2</td>
<td>96</td>
<td>76.7</td>
<td>15.9</td>
<td>136</td>
<td>S</td>
</tr>
<tr>
<td>G96-3</td>
<td>96</td>
<td>90.7</td>
<td>18.8</td>
<td>182</td>
<td>S</td>
</tr>
<tr>
<td>G144-1</td>
<td>144</td>
<td>121.9</td>
<td>16.8</td>
<td>64</td>
<td>S</td>
</tr>
<tr>
<td>G144-2</td>
<td>144</td>
<td>93.8</td>
<td>12.9</td>
<td>55</td>
<td>S</td>
</tr>
<tr>
<td>G144-3</td>
<td>144</td>
<td>109.9</td>
<td>15.2</td>
<td>66</td>
<td>S</td>
</tr>
</tbody>
</table>

G48-1= GFRP bar with 48 mm embedment length and specimen number 1
P=Pullout failure; S=Splitting failure

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Embedment length (mm)</th>
<th>Load $P_{\text{max}}$ (kN)</th>
<th>Average bond stress $\tau$ (MPa)</th>
<th>Free end slip (micron)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>PS120-1</td>
<td>120</td>
<td>24.8</td>
<td>4.1</td>
<td>1433</td>
<td>P</td>
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<tr>
<td>PS120-2</td>
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<td>21.5</td>
<td>3.6</td>
<td>1305</td>
<td>P</td>
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<td>PS120-3</td>
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<td>4.7</td>
<td>389</td>
<td>P</td>
</tr>
<tr>
<td>PS150-1</td>
<td>150</td>
<td>20.2</td>
<td>2.7</td>
<td>1664</td>
<td>P</td>
</tr>
<tr>
<td>PS150-2</td>
<td>150</td>
<td>17.2</td>
<td>2.3</td>
<td>-</td>
<td>P</td>
</tr>
<tr>
<td>PS150-3</td>
<td>150</td>
<td>30.0</td>
<td>4.0</td>
<td>1406</td>
<td>P</td>
</tr>
<tr>
<td>RS120-1</td>
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<td>44.8</td>
<td>7.4</td>
<td>5.98</td>
<td>S</td>
</tr>
<tr>
<td>RS120-2</td>
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<td>60.2</td>
<td>10.0</td>
<td>7.69</td>
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<td>150</td>
<td>75.8</td>
<td>10.0</td>
<td>12.20</td>
<td>S</td>
</tr>
<tr>
<td>RS150-2</td>
<td>150</td>
<td>64.2</td>
<td>8.5</td>
<td>2.5</td>
<td>S</td>
</tr>
</tbody>
</table>

PS=Plain Steel bar; RS=Ribbed Steel bar
ANALYSIS AND DISCUSSION

Failure mode and average bond strength

As can be seen in Table 3, splitting failure mode dominated the ribbed steel bar specimens due to the relatively low confinement and longer embedment lengths of the specimens. This failure mode was also observed for the GFRP bars with longer embedment lengths, i.e. 96 mm and 144 mm specimens. Tepfers (1979) reported that the splitting failure is caused by the radial component of the bond stress. This radial bond stress generates a splitting stress in the concrete around the reinforcement bar. When this stress exceeds the tensile strength of the concrete, it results in the splitting of the specimens. Yet this type of failure was not observed in the plain steel bars despite having similar or longer embedment lengths. This depicts the poor bond between the plain steel bar and the concrete which, unlike the ribbed steel bars and sand-coated GFRP bars is not enhanced by the ribs and the sand coating, respectively.

After failure, the GFRP bars showed a thin layer of concrete on their surface as shown in Figure 3 (a). In case of the ribbed steel bars on the other hand, the concrete is intact with no damage as the specimen splits; i.e. leaving no concrete on the bar surface (Figure 3 (b)). This indicates that the bond between the ribbed steel bar and the concrete is still undamaged at the point of concrete splitting. The plain steel bar slipped out of the concrete without damaging the interface as can be seen in Figure 3 (d), depicting its poor bond strength.

Splitting failure can be considered as premature because the bond has not yet attained its maximum value. Due to their ribs, the ribbed steel bars generate a higher splitting stress than the GFRP bar. This results in the splitting of the concrete earlier than the GFRP bar specimens. Indeed, as can be noticed, the GFRP reinforced GPC concrete specimens show an apparent higher bond strength than ribbed steel reinforced GPC concrete specimens. The average bond stress at failure for G144 specimens is 15.0 MPa, whereas that of the RS120 is only 8.7 MPa. Even at higher embedment lengths, where bond stress is known to decrease (Cosezna et al. 1997, Maranan et al. 2015, Tighiouart et al. 1998 Lee et al. 2013, Okelo and Yuan 2005), GFRP specimens still show a higher bond stress than ribbed steel bar specimens at the point of concrete splitting. This lower bond stress of ribbed steel bar compared to GFRP bars in case of splitting failure was also reported by Maranan et al. (2015).

In case of pull out failure on the other hand, the specimens failed due to interface failure. The ribbed steel specimens offer a better pullout resistance due to their ribbed surface. This results in a better bond strength for ribbed steel specimens. This was observed by Maranan et al. (2015) experimental results for specimens with equal embedment length of five times the bar diameter. For the pullout failure to occur, in case of GFRP, the sand coated bar need to slip over the concrete surface. The interlock between the bar and the concrete is provided by the sand coating and the concrete between the sand grains. Thus the bond strength is only as strong as this interface interlock and the confinement provided by the concrete. Once the sand grains or the concrete start shearing, pullout will occur. However, in case of ribbed steel bars the pullout failure occurs only when shear cracks are initiated in the concrete keys between ribs. As the ribs are stronger than the sand coating, the pullout strength of ribbed steel bars is higher than sand-coated GFRP bars.
Splitting failure and bond-angle

The bond stresses between a reinforcing bar and the concrete make an angle $\alpha$ with the bar axis as schematically shown in Figure 4. This angle mainly depends on the type of reinforcing bar used and controls the bond behaviour specifically the splitting failure. Higher bond-angles result in higher splitting stress on the concrete surrounding the reinforcement. Based on the bond angle, the stresses are divided into tangential ($\tau$) and radial ($\sigma_r$) components. The radial stress component generates a hoop/circumferential stress ($\sigma_t$) around the concrete covering the reinforcement (Figure 5).

![Figure 4 Radial components of the bond force balanced against tensile stress rings (Tepfers, 1979)](image)

The splitting of the concrete cover is a common bond failure mode in reinforced concrete structures. The hoop stress generated by the radial component of the bond stresses is resisted by the tensile strength of the concrete. Once the tensile strength is exceeded by the hoop stress, splitting failure occurs. When the bond-angle increases, the radial component of the bond stress increases, and this consequently facilitates the splitting failure mode. The splitting force depends mainly on the concrete cover thickness, bar diameter and type of bar. As can be seen in Tables 2 and 3, both the deformed steel bars and the GFRP bars have specimens with splitting failure. However, despite the similar cover and diameter for both steel and GFRP bars, the splitting bond forces are different. GFRP bars with splitting failure showed an average failure load of 97 kN, whereas ribbed steel bars showed a much lower value of 60 kN. The higher radial bond force generated by the ribbed steel bar caused this early failure of the specimen.

![Figure 5 Splitting failure of ribbed steel bar specimens; (a) Radial and tangential stresses (b) RS120-1 splitting failure](image)

Tepfers (1979) analysed the state of stress in concrete due to bond forces from ribbed reinforcing bars. The radial stresses from the bond were regarded as a hydraulic pressure on the concrete wall. The concrete was modelled in three different ways; uncracked elastic, partly cracked elastic and uncracked plastic. The derived equations for each of these models respectively are as described below:

\[
\sigma_t = \tau \tan \alpha \left( \frac{(c+d/2)^2+(d/2)^2}{(c+d/2)^2-(d/2)^2} \right) 
\]

\[
\sigma_t = 1.664\tau \tan \alpha \left( \frac{c+d/2}{c-d/2} \right) 
\]

\[
\sigma_t = \frac{d}{2c} \tau \tan \alpha 
\]

where $\sigma_t$ is tangential stress, $d$ is diameter of the bar, $c$ is the concrete cover (the smallest concrete cover around the reinforcement), $\tau$ is the bond stress and $\alpha$ is the angle the bond stress makes with the axis of the bar.
Splitting failure of the specimen occurs when the tangential stress equals the tensile strength of the concrete. For comparison, the bond angles are calculated by substituting all the known parameters into Eq. 2, Eq. 3 and Eq. 4. The tensile strength of the concrete is taken as the maximum tangential stress. Table 4 summarises these results.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Uncracked elastic</th>
<th>Partly cracked elastic</th>
<th>Uncracked plastic</th>
</tr>
</thead>
<tbody>
<tr>
<td>GFRP</td>
<td>15</td>
<td>25</td>
<td>50</td>
</tr>
<tr>
<td>Ribbed steel</td>
<td>28</td>
<td>47</td>
<td>71</td>
</tr>
</tbody>
</table>

For specimens with an approximate cover to diameter ratio of 2, Tepfers recommend using the average of partly cracked elastic and uncracked plastic models. For GFRP bars, the bond angle according to this recommendation becomes 37 degrees, whereas that of steel bars becomes 59 degrees. Tepfers and De Lorenzis (2003) also reported the smaller bond angle for sand-coated GFRP bars. In fact, they reported a bond-force angle of 35 degree for a sand-coated GFRP bars. They suggested that, the probable reason for the lower bond-angle is that the sand coated surface of the bar creates a softening layer with some ability to transfer tension in concrete close to the bar. The splitting tendency of a reinforcing bar increases with increasing of the bond-angle (Tepfers and Olsson, 1992), and thus the lower failure load of ribbed steel bars.

**Bond-slip curves**

For splitting failure, the bond-slip curve can be divided into two parts; the linear region and the nonlinear region. In addition to these regions, a softening branch is found in pullout failure specimens. Figure 6 shows different bond-slip curves for GFRP, plain and ribbed steel bar specimens. The GFRP bar specimens displayed a longer nonlinear region compared to the ribbed steel bars. This is due to the much brittle failure of the specimens with ribbed steel bars. Despite their ribbed bar counterparts, the plain steel bar specimens, displayed a longer nonlinear curve compared to the GFRP bar specimens. The small amount of slip observed at the free end of ribbed steel bar specimens further portrays the sudden brittle failure. The average slip recorded for both RS120 and RS150 specimens for instance is only 7 microns, which shows a negligible amount of bar displacement at the free end when the concrete splits.

![Figure 6 Bond-slip curves for GFRP, plain and ribbed steel bar specimens](image)

**CONCLUSIONS**

This paper has presented the comparison of bond properties of GFRP reinforced GPC concrete and steel reinforced GPC concrete. From the analysis, it was observed that ribbed steel bars have a better bond performance which was verified by their better bond strength in case of pullout failure, but a higher splitting tendency and thus lower bond stress at the point of splitting. The average bond stress at the point of splitting failure is about 16.4 MPa and 8.9 MPa, respectively for sand-coated GFRP bars and ribbed steel bars, showing the more brittle and sudden splitting of specimens with ribbed steel bars. Bond angles of 59 and 37 were determined for ribbed steel bars and sand-coated GFRP bars, respectively explaining the higher splitting tendency of ribbed steel bars.
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PULL-OUT BEHAVIOR OF CFRP GROUND ANCHORS WITH TWO-STRAP ENDS

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ABSTRACT
Pull-out experiments were performed on three carbon fiber-reinforced polymer (CFRP) ground anchors simulating their applications in different rock types which provided different confinement levels. The CFRP tendons comprised, on the ground side, a prefabricated conical anchor body of high-strength grout in which two CFRP straps were embedded. The anchors reached an average load-bearing capacity of 1384 kN with final failure occurring in the CFRP straps. Grout failure was successfully prevented by an adequate grout selection and installation of CFRP confinement rings to balance spreading forces at the strap ends. The confinement level provided by the surrounding media influenced the activation of the CFRP components in the anchor body by influencing the friction at the CFRP/grout interfaces.

KEYWORDS
Carbon fiber-reinforced polymer, ground anchor, two-strap end, confinement, pull-out behaviour.

INTRODUCTION
Fiber reinforced polymer (FRP) tendons are increasingly used to replace conventional steel tendons in ground anchors, taking advantage of their high strength-to-weight ratio and good corrosion resistance. The applications of AFRP and CFRP tendons, especially in permanent prestressed cases, have been reported since 1990 (Tokyo Rope Co. Ltd 2016). Conceptually similar to steel strands, FRP strands were also developed by twisting a certain number of small-diameter wires. A high load-bearing capacity could be achieved by forming a FRP cable assembled from several strands, for example in the Carbon Fiber Composite (CFCC) system commercialized by Tokyo Rope in Japan (Benmokrane et al. 1997, Tokyo Rope Co. Ltd 2016). FRP Cables with a similar concept, assembled from rods with nominal diameters of 4.0 mm (Wang et al. 2015) or 12.6 mm (Zhang et al. 2014), were also developed and studied.
To anchor the tendon on the air and ground sides, mechanical or bonded anchors are commonly used (Schmidt et al. 2012). However, two problems exist in these types of CFRP anchors, which could lead to a premature failure in the anchor and thus not allow to exploit the full tendon capacity: 1) high shear and through-thickness stress concentrations existing at the anchorage are critical due to the anisotropic properties of CFRP fibers; 2) uneven load distributions among the assembled strands or rods occur, i.e. some of them were obviously less loaded compared to others (Wang et al. 2015).

Figure 1 Prestressed and permanent CFRP ground anchor with multi-strap end
A simpler and much more material-tailored anchorage method, based on strap ends, was thus developed (Winistoerfer 1999). A new application of this strap anchorage method for permanent prestressed CFRP ground anchors was recently proposed (Fan et al. 2016), which consists of a CFRP tendon with a multi-strap end on the ground side, embedded in a prefabricated high-strength grout cylinder confined with CFRP rings, as shown Figure 1. The main purpose of the rings is to deviate the spreading forces at the embedded strap ends into the cylinder’s axial direction and not to increase the grout strength. The grout cylinder is thus stepwise axially loaded by the axial components of the spreading forces. The ground anchor with the prefabricated anchor body can be 20–80 m long and coiled, then transported to the construction site, inserted into the borehole, anchored by injecting fresh standard (normal-strength) grout and finally prestressed to 60% of the design load.

In a first stage, a CFRP ground anchor with a single-strap end on the ground side was developed (Fan et al. 2016). A CFRP confinement ring, according to the anchor concept (see Figure 1), was installed at the strap end to deviate the spreading forces. A load-bearing capacity of around 500 kN was achieved in pull-out experiments, which is well-tailored for soil applications.

This paper reports on a second development stage, where a two-strap end was conceived on the ground side to increase the load-bearing capacity to more than 1000 kN, which normally need a rock media to be anchored. Anchor specimens were pulled out from mortar cylinders confined by steel tubes of different thicknesses which simulated different confinement levels of the rock mass.

EXPERIMENTAL PROGRAM

CFRP ground anchor specimens

The CFRP tendons used in this study were produced by Carbo-Link, Fehraltorf, Switzerland, as shown in Figure 2. The tendons were composed of unidirectional UTS50 F24 24k 1600tex D carbon fibers impregnated with XB 3515 AD1571 ACC1573 epoxy resin; the fiber volume fraction was 60 ± 2%.

The two-strap end of the CFRP tendon was embedded in a 1060-mm-long high-strength grout body with inclined and corrugated surface on the ground side, as shown in Figure 3. A non-shrink sand/cement high-strength grout (SikaGrout-212 provided by Sika Schweiz AG, Switzerland) with a maximum aggregate size of 4 mm was used. Three 150-mm-long and 2-mm-thick identical CFRP confinement rings, consisting of the same materials as the tendon, were installed around the two strap ends and the division point. The anchor body geometry was designed to fit in boreholes with a minimum diameter of 130 mm.

The prefabricated anchor body was inserted into a 1200-mm-long steel tube, simulating a rock mass as shown in Figure 4. The tube was then filled with a fresh sand/cement grout (Sika normal rock anchor mortar, provided by Sika Schweiz AG, Switzerland) with a maximum aggregate size of 0.8 mm. Pull-out experiments were conducted on three anchor specimens with two different tube thicknesses to study the influence of the rock stiffness on the load-bearing capacity of the anchor; an overview of the experimental series is shown in Table 1.
Table 1 Summary of experimental results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Tube thickness (mm)</th>
<th>First peak (kN)</th>
<th>Second peak (kN)</th>
<th>$K_{exp}$ (kN/mm)</th>
</tr>
</thead>
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<tr>
<td>ST10-1</td>
<td>10</td>
<td>1344</td>
<td>925</td>
<td>92.0</td>
</tr>
<tr>
<td>ST10-2</td>
<td>10</td>
<td>1419</td>
<td>230</td>
<td>92.0</td>
</tr>
<tr>
<td>ST5</td>
<td>5</td>
<td>1389</td>
<td>1402</td>
<td>86.2</td>
</tr>
</tbody>
</table>

Experimental setup and instrumentation

The pull-out experiments were conducted on a Trebel 10MN machine, as shown in Figure 5. The anchor was suspended at the air side through a steel pin to the top fixed beam of the machine. The pulling load was applied to the top surface of the anchor body through a loading frame consisting of a bearing plate, two cross beams, two transverse reinforcements and 18 thread bars; the bars were fixed to the cross beams on one side and to the bottom movable beam of the machine on the other side. The loading was applied to the bottom movable beam in displacement-control mode at a rate of 1 mm/min. After four load cycles up to 100, 300, 500 and 700 kN respectively, the anchor was loaded up to failure.

![Figure 5 Experimental set-up (dimensions in [mm], DIC white-black paint on tube)](image)

The instrumentation layout is shown in Figure 6. Three types of instruments were used: Linear Variable Differential Transformer (LVDT) transducers, strain gages and Digital Image Correlation (DIC). Two LVDTs were symmetrically fixed on the rod at a distance of 115 mm from the top surface of the 50-mm-thick bearing plate to obtain an average pull-out displacement.
RESULTS AND DISCUSSIONS

Load vs pull-out displacement responses and failure modes

All specimens exhibited similar load vs pull-out displacement responses in the failure cycle up to a first peak, as shown in Figure 7. An almost linear response was observed, except a slight nonlinear behavior at the beginning; the latter was attributed to a progressive debonding at the CFRP/grout interface, which also caused a small residual displacement after the four initial load cycles up to 700 kN, as shown in Figure 7 for anchor ST5. The stiffness in the linear range, $K_{exp}$, assumed as being the slope of the linear segment between 500 and 1200 kN, was similar, as shown in Table 1; anchor ST5 exhibited only 6.3% lower stiffness than the ST10-1/2, which could be attributed to a slightly larger deformation due to the weaker confinement.

The three anchors reached similar loads at the first peak, indicating that the confinement level had low influence on the load-bearing capacity of the anchor. At this first peak, delamination was observed in the visible air-side straps in anchors ST10-1/2, while no damage was recognizable in ST5. After the first peak, the load dropped and increased again to a second peak in all specimens where partial or complete rupture of the air-side strap occurred in anchors ST10-2 and ST5 respectively, see Figure 8. The obtained peak loads are listed in Table 1.
Figure 8 Failure modes: (a) cut view of anchor ST10-1; details in (b) ST10-1, (c) ST10-2, (d) ST5
After failure, the specimens were cut into two halves in the strap plane, see Figure 8. All anchors exhibited rupture in the embedded CFRP tendons; no compression failure in the grout parts was observed, except small cracks located in the normal-strength grout at the end of the CFRP rings, see Figure 8 (a). In anchor ST10-1, the large and small straps were completely separated from the rod at the division point on one side, while partial rupture or delamination occurred in the semicircles of the small or large straps, see Figure 8 (b). In anchor ST10-2, complete rupture was visible at the division point of the large strap and in the semicircle of the small strap. Furthermore, partial rupture or delamination occurred in the semicircle of the large strap and the division point of the small strap, see Figure 8 (c). In anchor ST5, complete rupture occurred in the semicircle of the large strap and partial rupture in one straight part of the large and at the end of the small strap; no damage was visible at the division point.

Figure 9 Load vs tensile strain responses of ground-side large strap: (a) along straight segment; (b) at strap end
Load vs tensile strains in CFRP tendons

The load vs tensile strain responses in fiber direction of the embedded (ground-side) large and small straps are shown in Figs. 9 and 10 respectively. In all the anchors, the straps were activated progressively from the division point to the small and then to the large strap end due to the progressive loss of bond and subsequent friction at the CFRP/grout interface. The gages close to the division point (T7/8) thus started responding firstly, at lower loads (below 150 kN), while the gages around the large strap (T1/2/14) were activated lastly, at much higher load (above 350 kN).

The strap activation sequence also depended on the confinement level, i.e. at the higher confinement the strap was activated later due to the higher friction at the CFRP/grout interface. Accordingly, the semicircular parts of the large straps (positions T1/2/14) and small straps (T5/13) in anchor ST10-2 were activated later (at around 450 and 350 kN respectively) compared to ST5 (at around 380 and 270 kN respectively). After the activation, the strain rates in ST10-2 were also slightly lower than in ST5 due to the higher friction.

Due to the early activation, positions T7/8 generally exhibited the maximum strains. However, gages T4/7/8 stopped measuring after certain loads due to the propagating slippage at the CFRP/grout interface. The maximum strains were measured in the small strap of anchor ST5 due to the earlier activation compared to the large strap and lower confinement level; they approached the ultimate strain of the CFRP material. The strains in the large straps, however, remained far below the ultimate strain.

Furthermore, complete recovery of the strap deformation during unloading was prevented by the friction behavior as well, which also led to the residual displacements after unloading during the first four cycles, shown in Figure 8. The amount of strain recovery also depended on the strap location, i.e. more strain recovery was observed closer to the division point due to the smaller friction, see Figures 11 (b) and 12 (b).

CONCLUSIONS

Pull-out experiments were performed on three CFRP ground anchors with two-strap ends embedded in a prefabricated high-strength grout anchor body on the ground side. The anchor body was embedded in normal strength grout confined with steel tubes of different thicknesses to simulate the confinement of different rock masses.

The results showed that the targeted 1000-kN anchor capacity was reached. Failure occurred in the CFRP tendons at different positions on the embedded air- and ground- sides which proved an almost uniform use of the capacities of the different strap components. Furthermore, the conceptual elements to prevent grout failure, i.e. selection of high-strength grout in the anchor body, CFRP confinement rings to balance and deviate spreading forces and the complex conical anchor body shape to introduce the forces into the normal grout proved to be effective and well-tailored, grout failure did not occur.
ACKNOWLEDGMENTS

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REFERENCES


BOND MODELLING OF FRP REBARS IN FRC

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ABSTRACT

Existing bond models for FRP reinforcements are based on limited number of parameters only. Present study includes the description of an extensive experimental work, as well as an analytical part for modelling the bond behaviour of FRP rebars in plain and in fibre-reinforced concrete. Experimental parameters consisted of: (1) type of FRP rebar (carbon, glass or basalt fibres) including different surface characteristics and moduli of elasticity, (2) concrete strength and type (four different grades of conventional concrete and two SCC mixes), (3) type of fibres in concrete mixes (no fibres, steel, synthetic micro or synthetic macro polymer fibres), resulting 216 pull-out tests and 144 additional material property tests (concrete compressive strength tests and concrete splitting tensile strength tests). All bond test results were analysed for possible ways to refine bond modelling. Conclusions are drawn on bond behaviour and modelling by using the studied parameters.

KEYWORDS

FRP, carbon, glass, basalt, FRC, short fibres, bond behaviour, modelling of bond.

INTRODUCTION

Mild steel rebars have been extensively used for reinforced concrete structures as internal reinforcements owing to their numerous advantageous properties such as: ductility, high tensile strength, bendability etc. However, they present some disadvantages too being the most significant one is that steel reinforcement is susceptible to corrosion. One of the main advantages of Fibre Reinforced Polymer (FRP) reinforcing materials is their excellent corrosion resistance (Baena et al. 2009 and Boroszyó 2014).

Numerous studies can be found in literature underlining the significance of the bond behaviour of reinforcement in concrete. Yet, in case of FRP rebars there are still open issues owing to the high number of parameters which affect the bond.

During the last decades, numerous experimental studies have been conducted to investigate the bond strength of FRP rebars in concrete. The studies also investigated the influence of the parameters, such as rebar diameter, concrete strength, concrete cover, embedment length, surface treatment, etc. on the bond characteristics of FRP rebars (Achillides and Pilakoutas 2004, Baena et al. 2009 and Haffke et al. 2015). On the other hand, less attention was payed to determine analytically the bond stress-slip constitutive law for FRP rebars, which is essential for finite element analysis of FRP reinforced concrete structures. Therefore, only few FRP bond stress-slip models have been reported so far, and the suitability and capability of these models for numerical modelling of FRP reinforced concrete structures have not been confirmed yet.

One of the numerous bond affecting parameters that still calls for further investigation is the effect of short fibres, mixed into the concrete matrix. Mixing short fibres into the concrete matrix represents a possible way to confine the concrete in compression zone and provide it with additional strain capacity.

The placement of fibre reinforced concrete can be difficult due to clumping of the fibres. Moreover the use of poke vibrator can be difficult as well. These problems may be overcome by using self-compacting concrete which contains short fibres (Sólyom and Balázs 2016). Available studies show that FRC has improved tensile strength, strain capacity and ductility over normal concrete (Czoboly and Balázs 2015).
Combining fibre reinforced self-compacting concrete with FRP reinforcing bars holds major benefits, because the additional strain capacity allows an increase in flexural strength over the equivalent conventionally reinforced beams (Ibell et al. 2009).

An overview of the combined use of FRP rebars with FRC is provided in Sólyom et al. (2015) and will be briefly summarized herein.

In the research by Belarbi and Wang (2004) Glass Fibre Reinforced Polymer (GFRP) and Carbon Fibre Reinforced Polymer (CFRP) rebars were used. GFRP rebars had 12.7 and 25.4 mm in diameter, surface wrapped with helical fibre strand and additionally sanded afterwards. CFRP rebars were 12.7 mm in diameter, with smooth surface. The fibres added to the concrete mix were commercially available polypropylene short fibres, with maximum length of 57 mm. From the experimental data (27 pull-out specimens) it was concluded that the addition of polypropylene fibres did not increase the bond strength, but larger slips were recorded. The large slip values made the bond behaviour more ductile and the failure mode changed from splitting to pull-out. This is in contradiction with the results reported by (Ding et al. 2014). The authors claim, that the addition of short fibres to concrete can enhance the bond strength of GFRP rebars and also reduces the slip corresponding to the bond strength. Others (Won et al. 2008) have found also that the addition of short fibres to concrete increases the bond strength of FRP bars.

It can be concluded from the above review, that there are inconsistencies in the available results and further research is needed.

In this paper two bond stress versus slip models will be assessed. They show how the parameters of the models change according to the studied bond influencing factors, namely: FRP fibre type and surface characteristics, short fibre type and concrete compressive strength.

**EXPERIMENTAL STUDIES**

The pull-out test is the most frequently chosen method for comparing the bond behaviour of different FRP rebars in various concrete compositions. This test is a powerful tool to study the effect of different parameters on bond strength, owing to its simplicity and ease of application.

150 mm cubic moulds were used to manufacture the pull-out specimens. The bars were vertically placed in the centre of the moulds with 5Ø bond length in the lower part of the moulds (Ø - rebar diameter). Following the pouring, the specimens were left in the moulds under laboratory ambient conditions for one day. Thereafter, the concrete cubes were demoulded, marked and placed under water for 6 days. After this period of time the concrete specimens were taken out of water and kept in laboratory air until testing (mixed curing was applied).

The concrete pull-out specimens were placed into a metal frame and the FRP rebars were gripped by the test machine. This gripped portion was considered as the loaded end of the test specimen and the relative displacement between the FRP rebar and concrete was measured with three Linear Variable Differential Transducers (LVDT). At the other end, usually referred to as unloaded or free end, the slip was measured by one LVDT. Displacement controlled test was selected on the loading machine to capture post-peak behaviour. The load was applied to the rebar at a rate of 1 mm/min and measured with the electronic load cell of the testing equipment (Instron 600 kN). An automatic data acquisition system was used to record the data transmitted by LVDTs. Three nominally identical specimens for each configuration were tested.

In order to study the influence of concrete strength on bond development between FRP bars and concrete six different concrete grades were used (mean values of concrete compressive strength ranging between 27.7 and 91.9 MPa measured on three 150 mm cubic specimens). Three different type of FRP bars were used, carbon, glass and basalt. CFRP and GFRP bars had similar surface characteristics (sand coated) and diameter (9.5 mm) as well. The Basalt Fibre Reinforced Polymer (BFRP) bars were 12.7 mm in diameter and had a surface with helical wrapping. Three different short fibres were added to the concrete matrix: steel, synthetic micro and synthetic macro polymer fibres.

The symbols of concrete mixes consist of 3 characters, the first C or S stands for traditional and self-compacting concrete, respectively. The second character represents the concrete strength, the higher value of the character, the higher the concrete strength is. The third character mix refers to the type of short fibres in concrete, namely: 1 represents that no short fibres are mixed to concrete, while 2 refers to synthetic macro, 3 to steel and 4 to synthetic micro fibres. For example C11 represents traditional concrete prepared according to composition C1 without short fibres, while S23 stands for self-compacting concrete prepared according to composition S2 with steel fibres.
The main objective of this experimental series was to examine the effect of the short fibres if added to concrete matrix on the bond strength and bond behaviour of FRP bars. Nevertheless, other additional factors that need further research to better understand their final influence on bond behaviour were also considered. The parameters taken into consideration during this study are: short fibres, type of FRP rebar fibre (modulus of elasticity), rebar surface characteristics and concrete strength.

RESULTS AND DISCUSSION

The results of the experimental part of this study along with the conclusions is presented in the authors’ previous paper (Sólyom and Balázs 2016). In this paper authors focus on analytical modelling, more specifically on curve fitting parameters of two already available analytical models: modified BPE (mBPE) (Cosenza et al. 1997) and CMR (Cosenza et al. 1995) models. These models are discussed in some papers (e.g. Baena et al. 2009), therefore only the necessary equations will be presented herein.

To perform numerical and analytical analysis of the behaviour of reinforced concrete members and structures including the interaction between concrete and reinforcement to determine for example the anchorage length, the crack width, etc. an analytical model of the bond stress - slip constitutive law is necessary.

Table 6 Parameter fitting of modified BPE and CMR models

<table>
<thead>
<tr>
<th>Mix</th>
<th>mBPE (double branch) model</th>
<th>CMR model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ascending branch</td>
<td>Descending branch</td>
</tr>
<tr>
<td></td>
<td>α</td>
<td>SE</td>
</tr>
<tr>
<td>C01</td>
<td>0.20</td>
<td>0.12</td>
</tr>
<tr>
<td>C02</td>
<td>0.23</td>
<td>0.02</td>
</tr>
<tr>
<td>C03</td>
<td>0.23</td>
<td>0.01</td>
</tr>
<tr>
<td>C04</td>
<td>0.29</td>
<td>0.02</td>
</tr>
<tr>
<td>C11</td>
<td>0.16</td>
<td>0.00</td>
</tr>
<tr>
<td>C12</td>
<td>0.27</td>
<td>0.04</td>
</tr>
<tr>
<td>C13</td>
<td>0.31</td>
<td>0.04</td>
</tr>
<tr>
<td>C14</td>
<td>0.31</td>
<td>0.01</td>
</tr>
<tr>
<td>C21</td>
<td>0.27</td>
<td>0.02</td>
</tr>
<tr>
<td>C22</td>
<td>0.25</td>
<td>0.05</td>
</tr>
<tr>
<td>C23</td>
<td>0.29</td>
<td>0.03</td>
</tr>
<tr>
<td>C24</td>
<td>0.29</td>
<td>0.02</td>
</tr>
<tr>
<td>C31</td>
<td>0.27</td>
<td>0.10</td>
</tr>
<tr>
<td>C32</td>
<td>0.26</td>
<td>0.02</td>
</tr>
<tr>
<td>C33</td>
<td>0.32</td>
<td>0.08</td>
</tr>
<tr>
<td>C34</td>
<td>0.27</td>
<td>0.02</td>
</tr>
<tr>
<td>S11</td>
<td>0.32</td>
<td>0.02</td>
</tr>
<tr>
<td>S12</td>
<td>0.30</td>
<td>0.01</td>
</tr>
<tr>
<td>S13</td>
<td>0.32</td>
<td>0.07</td>
</tr>
<tr>
<td>S14</td>
<td>0.30</td>
<td>0.07</td>
</tr>
<tr>
<td>S21</td>
<td>0.27</td>
<td>0.01</td>
</tr>
<tr>
<td>S22</td>
<td>0.27</td>
<td>0.02</td>
</tr>
<tr>
<td>S23</td>
<td>0.36</td>
<td>0.02</td>
</tr>
<tr>
<td>S24</td>
<td>0.35</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Table 1 shows the results of curve fitting procedure for the selected two analytical models in case of BFRP rebars. The accuracy of each analytical model was evaluated by calculating the discrepancy (standard error) between the experimental bond stress values ($\tau_{exp}$) and the corresponding analytical predictions ($\tau$) for each registered slip value ($s$). Results, calculated by equation (Eq. 1), are presented in columns: 5, 9 and 16:
\[ SE = \sqrt{\frac{\sum_{i=1}^{n}(\tau_i - \tau_{\text{exp}})^2}{n}} \]  

where \( n \) is the number of experimental values registered during the pull-out test in case of one specimen (ascending or descending branch), while \( \tau \) is calculated as a dependant of the assessed analytical model.

In case of the Double Branch (mBPE) model (Cosenza et al. 1997) the equations for calculating the bond stresses are as follows (Eqs 2 and 3):

- ascending branch \((0 \leq s \leq s_m, \text{ where } s_m \text{ is the slip corresponding to the maximum bond stress, } \tau_{\text{max}})\)
  \[ \tau = \tau_{\text{max}} \left( \frac{s}{s_m} \right)^\alpha \]  
  (2)

- descending branch \((s_m \leq s)\)
  \[ \tau = \tau_{\text{max}} \left[ 1 - p \left( \frac{s}{s_m} - 1 \right) \right] \]  
  (3)

where \( \alpha \) and \( p \) are parameters to be calibrated on the basis of curve fitting of the experimental data.

- the third branch, where the bond stress is constant, it is not taken into consideration because it was not observed on the experimental bond stress vs. slip diagrams (most probably due to the limitation of LVDTs).

In case of CMR model (Cosenza et al. 1995) the bond stress can be calculated by using Eq. 4:

\[ \tau = \tau_{\text{max}} \left[ 1 - e^{-(s/s_r)} \right]^\beta \]  

(4)

where \( \beta \) and \( s_r \) are parameters to be calibrated based on curve fitting of the experimental data.

In case of each concrete mix three specimens were tested, their mean obtained test data is presented in Table 1, columns: 2, 6, 10 and 13 (defined by curve fitting of the analytical model). In column 4 the standard deviation of the three values (in case of parameter \( \alpha \)) are calculated using Eq. 5.

\[ SD = \sqrt{\frac{\sum_{i=1}^{n}(\alpha_i - \alpha_{\text{avg}})^2}{n-1}} \]  

(5)

Similarly, the same equation (Eq. 5) can be used to calculate the values in columns 8, 12, and 15, only the studied parameter needs to be changed accordingly.

To be able to study the effect of concrete strength on the curve fitting parameters, the mean values of each concrete composition are defined in columns 3, 7, 11 and 14.

Similarly to Table 1, tables for GFRP and CFRP rebars were prepared. However, because of the limitation of the paper they are not presented herein, only a part of the results is given in Figure 1.

Analysing Table 1 and Figure 1, the following observations can be made if attention is given to BFRP rebar, which is relatively new on the market.

mBPE model (ascending branch), parameter \( \alpha \):

- slightly increases with the addition of short fibres, which in turn represents a less stiff bond stress vs. slip diagram. As \( \alpha \) increases the ascending part of the diagram is approaching to a linear function.
- the highest increase is in case of synthetic micro and steel fibres, Table 1, column 2.
- slightly increases with the increase of the strength of concrete, which in turn represents a less stiff bond stress vs. slip diagram, Table 1, column 3.

mBPE model (descending branch), parameter \( p \):

- increases with the increase of the concrete strength, which represents a steeper descending branch of bond stress vs. slip diagram, Table 1, column 7.
- no clear tendency for the effect of short fibres can be observed, Table 1, column 6.

CMR model (ascending branch), parameter \( \beta \):

- slightly increases with the increase of the strength of concrete (C0 and S2 are exceptions), which represents a negligible stiffness increase of the ascending branch of bond stress vs. slip diagram, Table 1, column 11.
- no clear tendency for the effect of short fibres can be observed, Table 1, column 10.

CMR model (ascending branch), parameter \( s_r \):

- increases with the increase of the strength of concrete, which represents a less stiff ascending branch of bond stress vs. slip diagram, Table 1, column 14.
- decreases with the addition of short fibres, which represents a stiffer ascending branch of bond stress vs. slip diagram. The highest decrease is achieved with synthetic macro fibres, Table 1, column 13.

Since the scatter of these parameters in case of BFRP rebars is relatively low, generally valid parameters for this specific BFRP rebar can be estimated as the resultant of these values, which are presented in the last row of Table 1: \( \alpha = 0.28, \beta = 0.49 \) and \( s_r = 1.81 \). However, if we do not take into consideration the values of C0 concrete.
composition, which seem to be outsider, they become: \( \alpha=0.29, \beta=0.41 \) and \( s_r=2.12 \). These values offer a satisfactory agreement with the results reported by (Baena et al. 2009) \( \alpha=0.18 \) to 0.23, \( \beta=0.36 \) to 0.84 and \( s_r=0.04 \) to 0.06 for GFRP rebar with similar diameter and slightly lower modulus of elasticity. The surface treatment consisted of helical wrapping and sand coating. The significant difference in \( s_r \) can be explained by the fact that different slip values (at loaded and unloaded end, respectively) were considered in different studies. Slightly different results were reported by (Yoo et al. 2015) for these parameters: \( \alpha=0.18, \beta=0.49 \) and \( s_r=0.22 \) for a GFRP rebar with similar diameter and helically wrapped surface.

Analysing Figure 1 the following observations can be made. Values of parameter \( \alpha \) in case of CFRP rebars (filled symbols) are the highest which leads to the conclusion that the higher the modulus of elasticity, the higher the parameter \( \alpha \) is, since CFRP and GFRP have similar surface characteristics and diameters. Furthermore, parameter \( \alpha \) is higher in case of rebars with sand coated surface treatment when compared to rebars with helically wrapped surface. BFRP and GFRP rebars have similar modulus of elasticity and diameters, however \( \alpha \) parameters are lower in case of BFRP rebars (unfilled symbols) than in case of GFRP rebars.

![Figure 1 Parameter α for mBPE model](image1)

**Figure 1** Parameter \( \alpha \) for mBPE model (each symbol represents an average value of three specimens). C, G and B stand for CFRP, GFRP and BFRP, respectively. Re – reference concrete (no short fibre) and concrete with synthetic macro (Ma), steel (St) or synthetic micro (Mi) fibres

![Figure 2 Experimental and analytical bond stress vs. slip diagrams](image2)

**Figure 2** Experimental and analytical bond stress vs. slip diagrams, C13 and C14 concrete mixes

In Figure 2 comparisons between experimental and analytical results are presented. It is straightforward that both mBPE and CMR models are in good agreement with the experimental results (Figure 2, left), however CMR have higher accuracy (see also Table 1, columns 5 and 16). Nevertheless, in Figure 2, just where the local maximum point in the bond stress vs. slip diagram is more accentuated, the similarity between experimental and analytical results is smaller, though it is still acceptable. At low slip values, the analytical models slightly overestimate the bond stresses, while as the slip increases this tendency changes. It needs to be highlighted, even though there are
only two of the numerous studied bond slip laws are presented here, the rest of the results are similar to the ones presented in Figure 2. The local maximum points are fairly typical for this type of helically wrapped BFRP rebars. More details on this phenomenon and the possible reasons of it can be found in the authors’ previous paper (Sólyom and Balázs 2016).

Finally, please note that, the experimental values were taken for $\tau_{max}$ and $s_m$, alternatively these parameters could have been estimated by formulae available in literature.

<table>
<thead>
<tr>
<th></th>
<th>a</th>
<th>b</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP</td>
<td>-0.00019</td>
<td>0.0268</td>
</tr>
<tr>
<td>GFRP</td>
<td>-0.00013</td>
<td>0.0190</td>
</tr>
<tr>
<td>BFRP</td>
<td>-0.000062</td>
<td>0.0089</td>
</tr>
</tbody>
</table>

Table 2 Values for parameters defining the relation between $\alpha$ and concrete compressive strength

$\alpha = a f_c^2 + b f_c$ (6)

where $a$ and $b$ are parameters which need to be defined by a curve fitting procedure (Table 2). It is worth mentioning, that the values in Table 2 are based on a limited number of specimens and further data is needed for the validation of this equation and these parameters.

CONCLUSIONS

In the present paper the interfacial bond behaviour between three types of FRP rebars (Glass, Basalt and Carbon fibres) in six different concrete compositions have been analysed. Since the experimental results have been presented in a previous paper, here the focus was given to analytical modelling, more specifically parameter fitting of two already available analytical models, namely: modified BPE and CMR models. Results of 24 mixes (216 pull-out tests) were considered. Studied parameters: concrete compressive strength, FRP fibre type and surface characteristics and short fibre type. Based on the results of the presented study the conclusions can be presented as follows.

Parameter $\alpha$ (mBPE model) increases with the increase of the compressive strength of concrete in case of all the investigated FRP rebars (basalt, carbon and glass). However the effect of short fibres is not straightforward, but it seems that $\alpha$ increases with the addition of short fibres, which in turn provides a less stiff bond stress vs. slip diagram. Values of $\alpha$ are higher in case of sand coated FRP rebars when compared to helically wrapped surface. Furthermore, increase of modulus of elasticity of FRP rebars results in an increase in the values of parameter $\alpha$. Similarly, parameter $\beta$ (CMR model) slightly increases with the increase of the compressive strength of concrete in case of all investigated FRP rebars. Values of $\beta$ are higher for sand coated rebars when compared to helically wrapped ones. Moreover, increase of modulus of elasticity of FRP rebars results in an increase of the values of $\beta$.

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STATIC AND FATIGUE BOND BEHAVIOUR OF GFRP BARS AND CONCRETE

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ABSTRACT

The experimental investigation is intended to study the influence of some parameters on the quasi-static and the fatigue behaviour of the bond between glass fibre reinforced plastic (GFRP) rebars and concrete. This is an important aspect in FRP reinforced concrete structural elements and of relevant importance in thin reinforced concrete panels extensively adopted as façade or pavements. The pull-out test set-up with eccentrically positioned GFRP ComBAR® bar was adopted to measure the effect of two parameters on the bond mechanical features: thickness of the concrete cover and concrete mechanical properties. For cycling tests, another parameter, the maximum load in the cycle, was introduced to estimate the fatigue life under different load levels. Quasi-static tests showed that concrete compressive strength influence on bond properties is much more pronounced than that of the concrete cover. Cyclic tests at highest load level revealed that higher concrete quality attained longer fatigue life.

KEYWORDS

GFRP rebars, eccentric pull-out test, GFRP-concrete bond, fatigue.

INTRODUCTION

Constructing using newly developed materials presents new wave of modern aspirations in civil engineering society. Since first vanguard applications in last decades of XX century, new FRP (fibre reinforced polymer) reinforcement is nowadays present in many practical design codes, endeavouring to take more and more share in global usage of reinforcement for concrete structures. Existing design codes and guidelines treat well the requirements and checks for proper design of FRP reinforced structures, but still missing some parts or underestimating real material characteristics due to insufficient knowledge concerning certain issues. One of the most important issues related to reinforced concrete design is bond that makes secure and balanced transfer of forces from reinforcement to surrounding concrete. Bond of FRP reinforcement is still treated as area of many aspects to research about (Sólyom et al. 2015). One of the simplest, but still effective method for experimental assessing of bond properties is standard pull-out test, recommended by Rilem (RILEM TC9-RC 1994). With the aim of simulating better real conditions in structure, but also for estimating concrete splitting tendency, pull-out test with eccentric placement of the bar is recommended by fib Bulletin 10 (Fib Task Group Bond Models 2000). Eccentric pull-out test is conducted in this research for estimating the possibilities of application low concrete cover in combination with FRP reinforcement. This was particularly investigated for the sake of application in thin plate elements that are usually prefabricated and used as façade panels, pavement or layer of sandwich composite plates. Non-corrosive nature of FRP bars and prefabrication of plate elements enable maximal lowering of the concrete cover, leaving the only concern about correct transfer of forces through so formed bond. Low concrete cover stands in favor of FRP bars due to its softer surface, comparing to steel bars, leading to lower local stress concentrations in bond, delaying splitting of concrete cover (Tepfers and De Lorenzis, 2003). Stress distribution necessary for developing the ultimate splitting crack pattern highly depends on the type of bar surface. Helically wrapped and sand coated GFRP (Glass Fibre Reinforced Polymer) bars exhibited earlier cover cracking for higher concrete mechanical properties (Tepfers and De Lorenzis, 2003) and GFRP bars with trapezoidal ribs showed delayed concrete cover cracking, comparing to specimens reinforced with classical steel rebars (Weber 2005). The concrete cover determines the bond failure mechanism. Cover of one Ø (Ø = bar diameter) generates splitting failure mechanism, while cover of 2Ø or more, generates pullout or fracture of bar (Ehsani et al. 1996).
One of the main field of application of FRP reinforcement is in bridge engineering, due to plenty of problems these structures experienced that were caused by corrosion of steel reinforcement. As fatigue is considered to be important phenomenon in bridge design, particular attention should be dedicated to fatigue of bond in FRP reinforced structures. Different researches showed different bond strength before and after cyclic loading (Shield et al. 1997) (Ceroni et al. 2006) (Wang and Belarbi 2010) (Alves et al. 2011). Fatigue is considered more significant damage factor in steel, than in FRP, reinforced structures, due to higher stiffness of steel material causing higher damage of bond interface during cyclic loads (Shield et al. 1997) (Ceroni et al. 2006). The best fatigue bond properties were observed in case of GFRP rods with stiff external layer of polymer and large deformations in it to increase the adhesion (Katz 2000). Helical wrapping and sand coating of FRP rods considerably reduced the rod and bond strength to cyclic loading (Katz 2000) (Lee et al. 2009).

Although a plenty of work is done, considering static bond behaviour, and some general conclusions are present, there are still some unsolved related issues. Unlike static bond behaviour, fatigue behaviour of FRP/concrete bond is still quite unknown topic.

The influence of some parameters affecting the bond of GFRP bar and concrete is experimentally investigated in the present paper. The centric and eccentric pull-out set-ups with GFRP ComBAR® and steel bars of diameter 8 mm were adopted to measure the effect of: position of the bar regarding the element edge, concrete mechanical properties and, for fatigue test, the cyclic load level. This research gives overview on best FRP/concrete bond performances considering all mentioned input parameters.

MATERIALS

Unidirectional E-glass FRP rebars, named ComBAR® (ComBAR GFRP Reinforcement 2015), were adopted with diameter 8 mm. The GFRP rebars are produced by pultrusion technique with vinyl-ester resin. They have external ribbed surface cut into the bar after curing (Figure 1). The mechanical properties of the rebars in the direction of the bar axis are: tensile strength 1500 MPa; elastic modulus 60 GPa (Combar GFRP Reinforcement 2015). For the sake of comparison, tests with conventional steel ribbed bars (grade B500B) of the same diameter (8 mm) were carried out.

![Figure 1. Surface of the GFRP rebars ComBAR.](image)

In total, four different concrete classes were used, with following designations: C1 (C20/25), C2 (C30/37), C3 (C45/55) and C4 (C50/60). The experimentally measured average (of 12 tests for each class) compressive cubic strengths of the concretes are listed in Table 1.

<table>
<thead>
<tr>
<th>Concrete designation/class</th>
<th>Average cubic strength [MPa]</th>
<th>Standard deviation [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1 (C20/25)</td>
<td>23.3</td>
<td>1.1</td>
</tr>
<tr>
<td>C2 (C30/37)</td>
<td>38.9</td>
<td>2.7</td>
</tr>
<tr>
<td>C3 (C45/55)</td>
<td>56.3</td>
<td>2.8</td>
</tr>
<tr>
<td>C4 (C50/60)</td>
<td>62.3</td>
<td>4.2</td>
</tr>
</tbody>
</table>

EXPERIMENTAL SETUP

This experimental campaign uses typical centric as well as eccentric pull-out tests. The geometry of the latter was slightly modified comparing to suggestions of fib Bulletin 10. Centric specimens are cubic with the 200 mm side and eccentric specimens have dimensions 200x200x150 mm (see Figure 2). Height of the eccentric specimens is not the same as suggested by fib Bulletin 10, but approx. 6 times higher than embedment length, which is positioned in the top part of the specimen. Such layout helps to avoid impact of compressive stresses resulting from reaction of support due to pulling out of the bar. The specimens were casted with bars in the side position, orthogonal to the direction of concrete casting, to ensure flat even surface of the concrete block side that is relying on bottom steel plate of the testing frame. Thereby it was enabled transferring the pressure uniformly from concrete block to steel plate. The geometry of the eccentric specimen that was slightly modified from cubic shape to allow anchorage to the loading frame and avoid excessive movements during cycling loading. Specimen geometry and relative bar position is shown in Figure 2a,b, where l is the bar free length from the adhesion zone to the grip tabs, Ø is the bar nominal diameter and c is the concrete cover.
Position of bars in tests was centric or with cover of 10, 15 and 20 mm regarding the specimen side. The length of bond zone between bar and concrete was selected as 5Ø, inserting an aluminium pipe to prevent forming of bond on the remaining part of the bar. Three specimens were quasi-statically loaded for each combination presented in Table 2, which is also showing specimen designation (first letter stands for bar type (G=GFRP, S=Steel), followed by concrete class (C1 – C4, for corresponding concrete class as in Table 1) and bar position (c=centric, 20, 15 and 10 for corresponding concrete covers).

Table 2. Combinations of performed tests and specimen designation.

<table>
<thead>
<tr>
<th>Bar position/concrete cover</th>
<th>Concrete class</th>
<th>Bar type</th>
<th>Specimen designation</th>
</tr>
</thead>
<tbody>
<tr>
<td>centric</td>
<td>C1</td>
<td>GFRP</td>
<td>GC1_c</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>GFRP</td>
<td>GC2_c</td>
</tr>
<tr>
<td></td>
<td>C3</td>
<td>GFRP</td>
<td>GC3_c</td>
</tr>
<tr>
<td>20 mm</td>
<td>C1</td>
<td>GFRP</td>
<td>GC1_20</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>GFRP</td>
<td>GC2_20</td>
</tr>
<tr>
<td></td>
<td>C3</td>
<td>GFRP</td>
<td>GC3_20</td>
</tr>
<tr>
<td></td>
<td>C4</td>
<td>GFRP/Steel</td>
<td>GC4_20/SC4_20</td>
</tr>
<tr>
<td>15 mm</td>
<td>C4</td>
<td>GFRP</td>
<td>GC4_15/SC4_15</td>
</tr>
<tr>
<td>10 mm</td>
<td>C1</td>
<td>GFRP</td>
<td>GC1_10</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>GFRP</td>
<td>GC2_10</td>
</tr>
<tr>
<td></td>
<td>C3</td>
<td>GFRP</td>
<td>GC3_10</td>
</tr>
<tr>
<td></td>
<td>C4</td>
<td>GFRP/Steel</td>
<td>GC4_10/SC4_10</td>
</tr>
</tbody>
</table>

Fatigue tests were performed only for GFRP bar, for each combination of concrete C2 and C3 and cover 10 and 20 mm. The loading levels for cyclic loading were selected according to the average static strength assuming R = 60 and 70%, where R represents ratio of the maximum load in the cycle and the static bond strength (R=τ_{max}/τ_u). Ratio of the minimum and maximum force in each cycle was selected as 0.1. For each combination and load level at least three specimens were cyclically loaded. Pull-out experimental test setup is shown on Figure 2c. To allow dilatation of the concrete in the direction perpendicular to the bar, a PTFE sheet was positioned between the steel plate and the concrete specimen. The bars were gripped with a cylindrical sleeve. Two displacement transducers (LVDT) were placed on the top of the cube to continuously measure displacement: one of the bar and another of the concrete surface next to the bar (Figure 2c). The difference of these displacements gives the slip between the bar and the concrete. Static pull-out tests were displacement controlled, with rate of 1 mm/min. Cyclic loading was force controlled with 5 Hz frequency, while the displacements were recorded continuously with 300 Hz acquisition frequency. Tests were running until reaching bond failure or one million cycles. In the latter case, bar was pulled out quasi-statically to estimate the residual bond strength.

RESULTS AND DISCUSSIONS

Static tests

Considering centric pull-out specimens statically loaded, this experimental work involved specimens of three different concrete classes: C1, C2 and C3 and ComBAR® GFRP bars of diameter 8mm (with external ribbed surface). This bar and specimen type are very similar to the one used in research of Baena in 2009, which detected less damage in the bars and more in the concrete for lower concrete strengths, and vice versa. General rule was
‘the higher the concrete strength is, the higher the bond strength is’ (Baena et al. 2009). As stated in the technical description of the producer (Schoeck Bauteile GmbH August 2013), using ComBAR, the bond behaviour is controlled by strength of concrete for compressive strength <60MPa. The present research mainly confirmed cited observances, as presented in Figure 3a, b. The bond strength, defined as the maximum of the average shear stress $\tau_{\text{max}}$ on the bond surface, was increasing with increasing of concrete compressive strength (Figure 3a). Damage mode involved shearing off the concrete ribs and slight grooves visible on the bar surface (Figure 3c). Similar damage mode was observed for all the concrete classes considered.

Considering eccentric pull-out specimens statically loaded, experiments comprised all combinations of: four concrete classes (C1 – C4) and GFRP bar cover of 10 and 20mm. Beside, 15 mm cover was combined with GFRP bar and concrete C4. Steel bars were considered in concrete C4 with cover of 10 and 20 mm (see Table 2). Some results of the considered combinations are collected in Figure 4, comparing: the different concrete qualities and covers for the GFRP bar in Figure 4a; two concrete covers and the centric position of the GFRP bars in the three concretes (Figure 4b); the GFRP and steel bars for three covers in the same concrete (Figure 4c).

Results for GFRP bars don’t give significant difference in bond strength with increasing the concrete cover, for each concrete class (Figure 4a). Comparison of the eccentric and centric pull-out tests shows tendency of the bond strength to decrease with increasing the confinement for the lower concrete qualities (Figure 4b). The effect of
concrete cover on bond strength of this type of GFRP bars becomes less pronounced as concrete class increases. Other investigations are supposed to better clarify this trend.

More important, than value of cover, is concrete class that significantly improves bond characteristics (Figure 4a). GFRP and steel bars show comparable values of bond strength for 10 and 20 mm concrete cover (Figure 4c).

Figures 5a,b show typical bond shear stress vs. bar slip curves, for covers 10 and 20 mm, and concrete classes C1 – C3. Bond strength doesn’t have relevant variation increasing the cover, but different post peak behaviour was observed. The higher concrete cover shows a higher fracture energy, i.e. the area underneath the $\tau_{\text{max}}$-slip curves. Figure 5c shows comparison of steel and GFRP bars with covers 10 and 20 mm and concrete C4. The main difference is the highest area underneath the curves of GFRP, meaning a higher energy to have a complete separation of the bar and concrete. Bond failure modes observed for all concrete qualities were, as in (Tepfers and De Lorenzis 2003): splitting of the concrete cover and pulling out of the bar. Visual inspection showed that for concrete cover 10 mm splitting of the concrete occurs and for all other cases pull-out of the bar was recorded.

**Fatigue tests**

The estimated mean bond strength allowed to set the loading levels for fatigue test of certain combinations including: concretes C2 and C3, GFRP bar cover of 10 and 20 mm and centric bar. The first loading level was selected as $R = 60\%$ in order to get a reference for succeeding tests. All groups of specimens exhibited very good behaviour for this load level. None of the combinations failed after one million cycles. The second load level for fatigue test was $R = 70\%$. For this, bond failure of almost all specimens was obtained before one million cycles. Different fatigue life occurred for different specimen types, as presented in Figure 6.

![Figure 6. Fatigue life for load level R = 70%](image)

**Figure 6. Fatigue life for load level R = 70%.**

Selecting 10 mm concrete cover, significantly longer fatigue life is achieved by specimens of higher concrete class (GC3_10), about a difference of one order of magnitude comparing GC2_10 (Figure 6). For cover of 20 mm, more similar values of the average fatigue lives were obtained for both concrete classes. The specimens of concrete C2 with centric bar (GC2_c) exhibited very good fatigue behaviour at loading level $R=70\%$, showing the tendency to survive one million cycles even with such high value of cyclic load. However, due to insufficient data for this specimen type, it still remains issue to be reassessed in next investigations. Increasing the concrete quality, specimens with centric bar (GC3_c) showed a reduction of the fatigue life, that is consistent with the behaviour
recorded for the quasi static loading (see Figure 4b). After one million cycles without failure for the load level R=60%, quasi-static pull-out of the bar was performed. Resulting bond strength did not have a reduction, in some case an increase (see Figure 7), but in the experimental scatter band of the pre-fatigue quasi-static bond strength.

CONCLUSIONS

This experimental campaign was performed to clarify some issues regarding GFRP/concrete bond under quasi-static and fatigue loading. The main conclusions are:

- Reducing the thickness of the cover from 20 mm to 10 mm, for this type of the bar of diameter 8 mm, affects bonding conditions in different manner: it lowers bond fracture energy, but it does not lower the value of the bond strength. Moreover, the concrete compressive strength influence on bond properties is much more pronounced than that of the concrete cover.
- Specimens of all combinations did not fail after one million cycles of cyclic loading at R = 60%. Moreover, they showed an unchanged post-fatigue bond strength compared to the pre-fatigue quasi-static one.
- For cyclic loading with R = 70%, the higher concrete quality demonstrated a longer fatigue life with the lower concrete cover, that is consistent with the results of the quasi static pull out tests.

The results will be used as a reference for analytical and numerical modelling of bond behaviour as well as for further investigations using real size structural members.

ACKNOWLEDGMENTS

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RELIABILITY ANALYSIS OF ADHESIVELY BONDED CFRP-TO-STEEL DOUBLE-LAP SHEAR JOINTS WITH THIN OUTER ADHERENDS

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ABSTRACT

In this paper, a comprehensive database of experimental results of CFRP-to-steel double lap shear joints is compiled and a probabilistic analysis of the data is conducted. The compiled experimental results are compared with the bond strengths predicted by the widely adopted Hart-Smith model for thin adherends and the model uncertainty is characterized, for five popular structural epoxy adhesives and two types of surface preparation techniques. Considering the mechanical and geometrical uncertainties of constituent materials, two reliability-based approaches, First-Order Reliability Method (FORM) and Monte-Carlo Simulation (MCS), are used to calculate the resistance factor at a target reliability index of 3.5. It is found that these two approaches agree well and the resistance factor varies with adhesives, surface preparation techniques, and CFRP type. The importance vector of random variables reveals that the model uncertainty contributes most to the variation of the limit state function.

KEYWORDS

Adhesive, surface preparation, double lap shear, model uncertainty, reliability, bond strength.

INTRODUCTION

The use of carbon fiber-reinforced polymer (CFRP) to repair, rehabilitate and strengthen steel beams has been widely researched in recent years (Sen et al. 2001, Tavakkolizadeh and Saadatmanesh 2003; Rizkalla et al. 2008. Fernando et al. 2009; Teng et al. 2012; Colombi and Giulia 2012; Zhao 2014). Debonding is a key failure mode associated with CFRP strengthened steel beams (Liu et al. 2001; Tavakkolizadeh and Saadatmanesh 2003; Schnierch 2005; Dawood 2008). Experimental studies on adhesively bonded CFRP-to-steel double-lap shear (DLS) joints showed that the debonding load exhibits high variability compared with that of other failure modes such as steel yielding and CFRP rupture (Harris and Dawood 2012). The uncertainty of the bond strength stems from material variability, joint geometry, manufacturing process, and uncertainties inherent in the use of simplified models to predict debonding. However, this uncertainty has not been quantified and the resistance factor of CFRP-to-steel bonded joint design is unknown. Quantifying the uncertainty of the predicted bond strength from the widely-used Hart-Smith (1973) model would be beneficial for design (Wu et al. 2012). In this research a survey of the experimental results on CFRP-to-steel bonded DLS joint was conducted and a database of results was compiled. The database was used to quantify the uncertainty associated with different key parameters that influence the bond, including adhesive type, surface preparation technique, dead load-to-live load ratio, and modelling uncertainty through a reliability-based analysis. Similar techniques have been applied to FRP strengthened concrete structures (Plevris et al. 1995; Okeil et al. 2002; Val 2003), however, reliability analysis for FRP strengthened steel structures is still in the early stages.

RELIABILITY ANALYSIS

The reliability analysis was conducted for a selected design space in which the analytical results are applicable. The statistical characteristics of each random variable was quantified and the limit-state function was defined in order to apply reliability methods.

Design Space

The experimental database is comprised of 242 CFRP-to-steel DLS coupons form 14 studies (Fawzia et al. 2005; Bocciarelli et al. 2007; Lam et al. 2007; Yu 2008; Chiew et al. 2008; Bocciarelli et al. 2009; Fawzia et al. 2010;
Liu et al. 2010; Nguyen et al. 2011; Al-Zubaidy et al. 2012; Wu et al. 2012; Wu et al. 2013; Fawzia et al. 2013; Kim et al. 2013), all of which failed by debonding, i.e. specimens that failed by CFRP rupture were excluded. The joint configurations covered by these studies are listed in Table 1. For all the DLS joints studied here, the axial rigidity of the inner adherend is higher than the outer adherends and the outer adherends are thin enough to neglect the peel stress (Hart-Smith 1973). For this preliminary assessment of uncertainty, this simplification helps to exclude the influence of peeling stresses which add additional uncertainty.

**Statistical characteristics of design parameters**

The model uncertainty, $\xi_R$, defined by the ratio of the measured and predicted resistance, was evaluated to characterize the uncertainty associated with using the Hart-Smith model to predict the bond strength. Material and geometry properties from the literature were used to calculate the bond strength for different joint configurations the measured and predicted bond strengths were compared. Therefore, the mean and coefficient of variation (COV) of the model uncertainty, $\mu(\xi_R)$ and COV($\xi_R$), for different adheres and surface preparation techniques can be calculated, as shown in Table 1. A Chai-squared test (Ang and Tang 2006) shows that the Weibull distribution represents the statistical distribution of the bias well.

<table>
<thead>
<tr>
<th>Design Scenario*</th>
<th>Number of specimens</th>
<th>$\mu(\xi_R)$</th>
<th>COV($\xi_R$)</th>
<th>$E_{\text{CFRP}}$ (GPa)</th>
<th>$t_{\text{CFRP}}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-SB-WL</td>
<td>27</td>
<td>2.149</td>
<td>0.228</td>
<td>83.9</td>
<td>1.468</td>
</tr>
<tr>
<td>A-SB-PL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A-AG-WL</td>
<td>21</td>
<td>2.436</td>
<td>0.186</td>
<td>83.9</td>
<td>1.468</td>
</tr>
<tr>
<td>A-AG-PL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S-SB-WL</td>
<td>19</td>
<td>4.557</td>
<td>0.263</td>
<td>88.7</td>
<td>1.468</td>
</tr>
<tr>
<td>S-SB-PL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S-AG-WL</td>
<td>33</td>
<td>3.980</td>
<td>0.255</td>
<td>88.7</td>
<td>1.468</td>
</tr>
<tr>
<td>S-AG-PL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TS-AG-PL</td>
<td>31</td>
<td>1.227</td>
<td>0.406</td>
<td>479</td>
<td>1.450</td>
</tr>
<tr>
<td>TT-AG-PL</td>
<td>38</td>
<td>2.032</td>
<td>0.390</td>
<td>479</td>
<td>1.450</td>
</tr>
<tr>
<td>MB-SB-WL</td>
<td>19</td>
<td>2.032</td>
<td>0.651</td>
<td>77.6</td>
<td>1.495</td>
</tr>
<tr>
<td>MB-AG-WL</td>
<td>54</td>
<td>2.704</td>
<td>0.130</td>
<td>77.6</td>
<td>1.495</td>
</tr>
</tbody>
</table>

Note: *A=Araldite 420; S=Sikadur 30; TT=Tyfo TC; TS=Tyfo S; MB=MBrace Saturant; SB=sand-blasted steel surface; AG=angle-ground steel surface; WL=wet-layup CFRP; PL=pultruded laminate.

The statistical characteristics of all the random variables are summarized in Table 2. The adhesive layer thickness ($t_a$) was treated as a random variable, and the thicknesses of pultruded CFRP laminate ($t_{\text{CFRP}}$) and steel plate thickness ($t_{\text{steel}}$) were treated deterministically. The thickness of the wet-layup CFRP laminate ($t_{\text{CFRP}}$), however, was treated as a random variable (Atadero et al. 2005). The statistical characteristics of the adhesive properties (shear strength $\tau_p$, shear modulus $G_s$, plastic shear strain $\gamma_p$) were assumed to be similar to another paste epoxy structural adhesive (Bresson et al. 2013). The moduli of pultruded CFRP laminate ($E_{\text{CFRP}}$) and steel plate ($E_{\text{steel}}$) were treated as deterministic variables. For wet-layup CFRP, however, the modulus ($E_{\text{CFRP}}$) was treated as a random variable (Atadero et al. 2005). The statistical characteristics of the loads (dead and live load uncertainty, $\xi_{DL}$ and $\xi_{LL}$) came from literature (AASHTO 2001, Allen et al. 2005).

<table>
<thead>
<tr>
<th>Random variable</th>
<th>Bias $\mu$</th>
<th>COV</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\tau_p$</td>
<td>1.000</td>
<td>0.048</td>
<td>Weibull</td>
</tr>
<tr>
<td>$G_s$</td>
<td>1.000</td>
<td>0.024</td>
<td>Lognormal</td>
</tr>
<tr>
<td>$\gamma_p$</td>
<td>1.000</td>
<td>0.198</td>
<td>Weibull</td>
</tr>
<tr>
<td>$t_a$</td>
<td>1.000</td>
<td>0.098</td>
<td>Lognormal</td>
</tr>
<tr>
<td>$E_{\text{CFRP}}$</td>
<td>1.000</td>
<td>0.095</td>
<td>Weibull</td>
</tr>
<tr>
<td>$t_{\text{CFRP}}$</td>
<td>1.000</td>
<td>0.044</td>
<td>Normal</td>
</tr>
<tr>
<td>$\xi_R$</td>
<td>Varies*</td>
<td>Varies*</td>
<td>Weibull</td>
</tr>
<tr>
<td>$\xi_{DL}$</td>
<td>1.050</td>
<td>0.100</td>
<td>Lognormal</td>
</tr>
<tr>
<td>$\xi_{LL}$</td>
<td>0.954</td>
<td>0.406</td>
<td>Lognormal</td>
</tr>
</tbody>
</table>

a. See in Table 1.
Limit State Function

The methodology for reliability analysis follows the similar procedure as reported in NCHRP Report No. 368 (Nowak 1999). The limit state function considering the variability of the design parameters is written as

\[
g(X) = \xi_R \left( \frac{\gamma_p^2 + 2\gamma_p \gamma_d}{\gamma_d} \right)^{n_p} \left( 1 + \frac{2E_{CFRP}t_{CFRP}}{E_{steel}t_{steel}} \right) - \frac{\phi R_n}{\gamma_{DL} + \gamma_{LL} \gamma_L} (\zeta_{DL} + \zeta_{LL} \gamma_{LL})
\]

Where, \( \phi \) is the resistance factor, \( R_n \) is the nominal resistance, \( \gamma_d \) is the live load to dead load ratio, and \( \gamma_{DL} \) and \( \gamma_{LL} \) are the load factors for dead load and live load, respectively.

RESULTS AND DISCUSSIONS

The importance vectors, which represent the relative contribution of different random variables to the variation of the limit state function, for four selected design scenarios are shown in Table 3. The other design scenarios generally follow the same trend as the ones presented here. Among them, the model uncertainty, \( \xi_R \), has the greatest influence on the overall uncertainty. Among the material properties, the CFRP modulus has the highest importance value, and the adhesive shear strength and plastic shear strain contribute more than does the adhesive shear modulus. The shear modulus is a demand type variable since it influences the formation of shear stress concentrations near the plate end where debonding initiates. The adhesive layer thickness has the least impact on the reliability index compared to other random variables, but this may differ if the peel stress is considered.

<table>
<thead>
<tr>
<th>Design Scenario</th>
<th>( \tau_p )</th>
<th>( G_\alpha )</th>
<th>( \gamma_p )</th>
<th>( t_s )</th>
<th>( \xi_R )</th>
<th>( E_{CFRP} )</th>
<th>( t_{CFRP} )</th>
<th>( \zeta_{DL} )</th>
<th>( \zeta_{LL} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-SB-WL</td>
<td>-0.055</td>
<td>0.018</td>
<td>-0.038</td>
<td>0.000</td>
<td>-0.951</td>
<td>-0.069</td>
<td>-0.031</td>
<td>0.060</td>
<td>0.285</td>
</tr>
<tr>
<td>A-AG-WL</td>
<td>-0.070</td>
<td>0.023</td>
<td>-0.047</td>
<td>0.000</td>
<td>-0.915</td>
<td>-0.088</td>
<td>-0.039</td>
<td>0.069</td>
<td>0.375</td>
</tr>
<tr>
<td>MB-AG-WL</td>
<td>-0.017</td>
<td>0.000</td>
<td>-0.009</td>
<td>0.000</td>
<td>-0.996</td>
<td>-0.020</td>
<td>-0.010</td>
<td>0.022</td>
<td>0.079</td>
</tr>
</tbody>
</table>

Considering a target reliability index to 3.5, the resistance factors for different design scenarios were calculated by FORM analysis and MCS for different live load-to-dead load ratios (\( Q_{LL}/Q_{DL} \)) and different types of CFRP (pultruded vs wet-layup), as shown in Figure 1. The FORM results agree well with MCS results in general, and the discrepancy between the two is less than 6%.

Larger adhesive ductility is preferred to achieve high bond strength, however, the material ductility might not be realized if, failure occurs at the bonded interface due to poor surface preparation. The results in Figure 1 highlight the complex interaction between adhesive type and surface preparation techniques. As shown in Figure 1(e), the resistance factor is very low for Tyfo TC adhesive with angle-grinding surface preparation, although this is the most ductile adhesive among the five adhesives considered. In contrast, Sikadur 30 is brittle but the corresponding resistance factor is above 0.75 for all design cases, regardless of surface treatment techniques and load ratios.

As shown in Figure 1(a), (c) and (g), sand-blasting consistently gives resistance factors greater than 0.6, and typically above 1.0 for these three adhesives: Araldite 420, Sikadur 30 and MBrace Saturant. However, surface preparation using angle-grinding technique gives inconsistent resistance factors. For example, for Tyfo TC, Tyfo S, and MBrace Saturant, angle-grinding led to resistance factors, \( \phi \), of less than 0.1, indicating high uncertainty in the results. In contrast, for Sikadur 30 and Araldite 420, angle-grinding resulted in resistance factors that were typically greater than 0.9.

In most LRFD-based structural design codes, live loads and dead loads are treated differently. Since double-lap shear coupons are not explicitly designed for a specific set of loads, different live load-to-dead load ratios were considered to illustrate the effect of this parameter on the reduction factor (and reliability) of steel-to-CFRP bonded joints as illustrated in Figure 1. Live load-to-dead load ratios up to 4 were considered to represent the reasonable range of values in typical structural applications. The resistance factor, \( \phi \), typical varies by about 0.2 at low live load-to-dead load ratios and typically converges for moderate to higher live-to-dead load ratios. Considering that the dead load of metallic structures typically represents a smaller portion of the total load, the asymptotic values at higher live load-to-dead load ratios may be more appropriate for typical designs, although the more conservative lower bound values should be used if the live load-to-dead load is low, or if it is unknown and could be low.
Figure 1 Resistance factors for different design scenarios

(a) Araldite 420 with sand-blasting
(b) Araldite 420 with angle-grinding
(c) Sikadur 30 with sand-blasting
(d) Sikadur 30 with angle-grinding
(e) Tyfo TC with angle-grinding
(f) Tyfo S with angle-grinding
(g) MBrace Saturant with sand-blasting
(h) MBrace Saturant with angle-grinding
Implications of constant resistance factor $\phi = 0.65$

In order to study the effects of a constant resistance factor on reliability indices, a resistance factor of 0.65, which is consistent with many brittle failure modes in North American codes and standards, was selected to compare the reliability indices for different design scenarios. Due to the significant impact of surface preparation techniques on the bond strength, the design scenarios were grouped by different surface preparation techniques, as shown in Figure 2. It was found that this resistance factor yields reliability indices above 3.0 for all design scenarios if sand-blasting is used, as shown in Figure 2(a). By a similar analysis if a target reliability of 3.5 needs to be strictly met for all design scenarios, one should use a resistance factor of 0.5. Angle-grinding yielded lower reliability indices except for Araldite 420 and Sikadur 30 adhesives, as shown in Figure 2(b).

![Figure 2: Reliability indices for different surface treatment techniques (a) sand-blasting and (b) angle-grinding](image)

**SUMMARY AND CONCLUSIONS**

This paper studied the reliability of adhesively bonded CFRP-to-steel DLS joints with thin outer adherends, for different design scenarios. The major findings of this paper are:

- The modeling uncertainty is highly influenced by the design scenario being considered and it is the most important random variable affecting the reliability index. This suggests that developing accurate and robust bond models can have a significant impact on the reliability of bonded joints in design applications and could dramatically influence the resistance factors that are used for bond.
- The adhesive shear strength and plastic shear strain have a greater influence on the bond reliability than the adhesive shear modulus. However, large adhesive ductility does not necessarily lead to high resistance factors due to the effect of surface preparation on the bond behavior.
- Surface preparation using sand-blasting gives consistently high resistance factors for the cases considered in this study, suggesting a lower degree of uncertainty. Angle-grinding led to inconsistent reliability for different cases.
- For Araldite 420, MBrace Saturant and Sikadur 30 using sand-blasting, a resistance factor of 0.65 gives reliability indices greater than 3.0 while a resistance factor of 0.5 gives reliability indices greater than 3.5.

**ACKNOWLEDGEMENTS**

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EVALUATION OF FATIGUE DURABILITY OF ADHESIVELY BONDED JOINTS BETWEEN STEEL PLATE AND CFRP LAMINATES

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ABSTRACT

In this paper, the fatigue durability of the adhesively bonded joints between steel plate and CFRP laminates were investigated experimentally aiming at the debonding in the end of CFRP laminates. Fatigue tests were conducted using the coupon specimens composed of the steel flat bar adhesively bonded by multi-layered CFRP laminates varying the stress range and the number of layers of CFRP laminates. The debonding lengths were measured utilizing the crack gauges attached at adhesively bonded joints. The result indicated that the fatigue life of adhesively bonded joints can be evaluated and its regression line equation can be proposed in the function of principle stress range independently to the number of layers of CFRP laminates.

KEYWORDS

CFRP laminates, adhesive joint, debonding propagation, fatigue durability.

INTRODUCTION

Recently, externally bonded patch plates, especially CFRP laminates, have proven to be effective for the application of repairing or strengthening the steel structures (JSCE 2012; JSCE 2013; Zhao 2013). However, there are problems remain for the design method of steel structure repairing or strengthening by CFRP laminates. One of the major points of concern in the use of this method is the adhesively debonding from the end of CFRP laminates which usually occurs ahead of the yielding of steel members or CFRP laminates (Lin et al. 2011; Nakamura et al. 2005). Moreover, although a number of studies have focused on bond behavior under static loading, there is a gap in understanding and lack of data under fatigue loading (Bocciarelli et al. 2009; Colombi et al. 2003; JSCE 2012; JSCE 2013; Lin et al. 2009; Liu et al. 2005).

The occurrence of the shear stress and normal stress simultaneously can be considered as the cause of the adhesively debonding from the end of CFRP laminates since there is a sudden change of cross-section at the end of CFRP laminates (JSCE 2013). Yet, there is no design recommendations related to this issue in Japan. In addition, to evaluate the fatigue durability of the repaired or strengthened steel structures by CFRP laminates, it is very essential and necessary to clarify the estimation method of the debonding propagation of the adhesive.

In this paper, toward the establishment of repair or strengthen method of steel structures by CFRP laminates, the fatigue strength and debonding progress of adhesively bonded joints under cyclic loading are experimentally evaluated.

EXPERIMENTAL PROCEDURES

Specimen Geometry and Materials

Figure 1 shows the shapes and dimensions of the steel plate and CFRP laminates adhesively bonded joint specimen. It consisted of a steel plate (500×50×9 mm) adhesively bonded by two CFRP laminates (300×50×1.2 mm per layer) at both sides of the steel plate. Here, the length of the CFRP laminates was set to 300 mm due to the limitation of the fatigue testing machine’s grip distance, and to make sure that there is no occurrence of the shear stress at steel plate’s center part. Two-pack type ambient-curable (cold hardening type) epoxy resin of Konishi E250 was used.
as adhesive. The thickness of the epoxy resin was controlled to approximately 0.4 mm. Table 1 presents the material properties of steel plate, epoxy resin and CFRP laminates. Here, the orthotropy was taken into account for the elastic modulus, Poisson’s ratio and modulus of rigidity values of CFRP laminates, (1: longitudinal direction, 2: width direction, 3: thickness direction).

Table 1 Material properties of steel plate, epoxy resin and CFRP laminates

<table>
<thead>
<tr>
<th>Materials</th>
<th>Elastic modulus $E$ (GPa)</th>
<th>Poisson’s ratio $\nu$</th>
<th>Shear Elastic Modulus $G$ (MPa)</th>
<th>Yield Strength $\sigma_y$ (MPa)</th>
<th>Tensile Strength $\sigma_{tu}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel plate(SS400)</td>
<td>200</td>
<td>0.3</td>
<td>76.9</td>
<td>320</td>
<td>453</td>
</tr>
<tr>
<td>Adhesive(E250)</td>
<td>1.5</td>
<td>0.3</td>
<td>0.58</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>CFRP laminates (High-strength type)</td>
<td>$E_{11}=150$</td>
<td>$\nu_{12}=0.34$</td>
<td>$G_{12}=5.2$</td>
<td>$E_{22}=8$</td>
<td>$\nu_{23}=0.05$</td>
</tr>
</tbody>
</table>

**Design of Specimen**

As mentioned above, the debonding usually takes place and progresses from end of the adhesive and CFRP laminates due to the occurrence of high shear stress and normal stress. Therefore, in this study, the fatigue strength of the adhesive was evaluated utilizing the principle stress $\sigma_{ep}$, which is the combination of the shear stress $\tau_e$ and normal stress $\sigma_e$. The principle stress $\sigma_{ep}$ at the middle thickness of the adhesive can be calculated using the following equation (Eq. 1).

$$\sigma_{ep} = \frac{\sigma_e}{2} + \sqrt{\frac{\sigma_e^2}{2} + \tau_e^2}$$  \hspace{1cm} (1)

where $\sigma_e$ and $\tau_e$ are, respectively, the normal stress and shear stress of adhesive. In this paper, $\sigma_e$ and $\tau_e$ were analytical obtained using Finite Element Analysis program (MSc. Marc2013). The appropriate number of mesh refinement of the thin adhesive joint in order to obtain the accurate stress values were considered.

In order to investigate the debonding progress at the end of adhesive, it is necessary to decide the appropriate stress value occurs at the end of adhesive. Regarding to the results of static loading tests with the high-strength type of CFRP laminates as presented in Table 1, there are some cases which the debonding did not occur or progress although the steel have already reached the yield point (Tezuka et al. 2012). In this study, therefore, the number of layers of CFRP laminates were raised to make the incensement of principle stress at the end of adhesive. Thus, 3 cases, CFRP with 1 layer, 5 layers and 7 layers, were selected to evaluate in this experiment. The $\xi_s$ of specimens with 1 layer, 5 layers and 7 layers were, respectively, 0.83, 0.50 and 0.45, which were calculated using the following equation (Eq. 2).

$$\xi_s = \frac{E_{Is}}{E_{Is} + E_{It}}$$  \hspace{1cm} (2)

where the subscript $s$ and $c$ refer to steel plate and CFRP laminates, respectively, and $t$ is the thickness.

Table 2 shows the conditions of the fatigue test. Taking into account the capacity of the fatigue testing machine, 5 cases (100, 120, 140, 160, 180 MPa) of nominal stress range $\Delta \sigma_{sn}$ were chosen. The stress ratio $R$ is set to 0.1 in all cases. Thus, from all 3 cases of different number of layers of CFRP laminates combining together, the principle stress range $\Delta \sigma_{ep}$ is from 12.5–33.5 MPa.
### Table 8 Experimental conditions

<table>
<thead>
<tr>
<th>Series of Experiments</th>
<th>Nominal stress of steel plate $\sigma_{sn}$ (MPa)</th>
<th>Principle stress of adhesive $\sigma_{ep}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Max</td>
<td>Min</td>
</tr>
<tr>
<td>CL1N180</td>
<td>200.0</td>
<td>20.0</td>
</tr>
<tr>
<td>CL5N100</td>
<td>111.1</td>
<td>11.1</td>
</tr>
<tr>
<td>CL5N120</td>
<td>133.3</td>
<td>13.3</td>
</tr>
<tr>
<td>CL5N140</td>
<td>155.6</td>
<td>15.6</td>
</tr>
<tr>
<td>CL5N160</td>
<td>177.8</td>
<td>17.8</td>
</tr>
<tr>
<td>CL5N180</td>
<td>200.0</td>
<td>20.0</td>
</tr>
<tr>
<td>CL7N100</td>
<td>111.1</td>
<td>11.1</td>
</tr>
<tr>
<td>CL7N120</td>
<td>133.3</td>
<td>13.3</td>
</tr>
<tr>
<td>CL7N140</td>
<td>155.6</td>
<td>15.6</td>
</tr>
<tr>
<td>CL7N160</td>
<td>177.8</td>
<td>17.8</td>
</tr>
<tr>
<td>CL7N180</td>
<td>200.0</td>
<td>20.0</td>
</tr>
</tbody>
</table>

#### Specimen Preparation

The CFRP laminates with 1, 5 and 7 layers were prepared in advance utilizing the same epoxy resin (E250). The average thickness of each adhesive layers of CFRP laminates is approximately 0.1 mm. Before bonding between steel plate and CFRP laminates, the surfaces of steel plate were blasted by alumina (#40), and the surfaces of CFRP laminates were sanded by sand papers (#150). The thickness of adhesive layer between steel plate and CFRP laminates were managed to 0.4 mm using the 0.4 mm glass beads (Jiang et al. 2006). After debonding, the specimens were cured at at 40 °C for 24 hours.

#### Test Setup and Experimental Conditions

The specimens were subjected to cyclic load with the frequency of $f=10–18$ Hz (high for the large stress range and low for the small stress range) as presented in Figure 2. The fatigue testing machine used in the experiment was electro-hydraulic servo type material strength testing machine (Shimadzu Servo Pulsar EHF-UB100kN). The friction welding high-strength bolt M16 was used for the grip section. The length of each grip sections was set to 70 mm. The lower part was fixed, and the upper part is allowed to act the tensile load. The waveform of the load was a sine wave, and the load, displacement and each strain values were measured at 5/10000 seconds interval using the dynamic strain-meter (Keyence NR-600). The series of tests were carried out at room temperature (approximately 15 °C).

To obtain the uniform of tensile stress, the strain gauge was applied at both sides of steel plate at the position of 15 mm from the grip section, and both sides of which crack gauge is applied. The setup position of the specimens were readjusted in case the variation of the strain value was large.

Furthermore, the below end of the CFRP laminates was fixed by fixture (Figure 2) in order to control and observe the debonding progress at the upper end of the CFRP laminates. Regarding to the existence of fixture, it should be noted that the influence of the presence or absence of fixture was already investigated.

![Fixture](image1.png) ![Overall view](image2.png) ![Crack gauge](image3.png)

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Figure 2 Specimen setup
Debonding propagation measurement method

Crack gauges were determined to measure the debonding length in the experiments of this research. The crack gauges used were KV-25B manufactured by Kyowa Electronic Instruments that have the measurement capacity up to 25 mm with 1 mm intervals. The crack gauges were attached on both surfaces of the adhesive end part (upper). The debonding lengths were measured due to the cutting (disconnection) of the crack gauges’ grid lines at the same time along with the occurrence of the debonding of adhesive.

The crack gauge was attached at the position of 2 mm from the adhesive end in the direction toward the centre of the specimen. Therefore, the number of cycles at initial debonding refers to the number of cycles when the debonding tip reached the position of 2 mm from the adhesive end in all. This was applied to all cases of specimens. The debonding lengths were measured from 2 mm to 27 mm at 1 mm intervals. It should be noted that strain gauges (length of 3 mm) were also installed to the centre part of CFRP laminates at the position of 5 mm from the CFRP laminate end.

RESULTS AND DISCUSSIONS

Table 3 presents the result of each specimens, the number of cycles from the test start (debonding length of 0 mm) to the debonding length of 2 mm, from the debonding length of 2 mm to 27 mm, and from the start to the debonding length of 27 mm. As described above, the initial fatigue life was considered as the number of cycles from the test start until the debonding length reached 2 mm.

From the result in Table 3, in the case of specimen of CFRP laminates with 1 layer (CL1FT), with the nominal stress of range $\Delta\sigma_{\text{sn}}=180$ MPa, there was no sign of debonding progress from the adhesive end or changes of strain value until the number of cycles reached $10^7$ times. CL1FT180 can be considered as the fatigue limit. However, this matter will be discussed later by comparing to the other test results. For the series of CFRP laminates with 5 and 7 layers, the debonding propagation from the adhesive end can be confirmed in every specimens. It should also be noted that in each case of experiments, the strain values of steel plates did not reach the yielding point.

Table 9 Test results (number of cycles)

<table>
<thead>
<tr>
<th>Series of Experiments</th>
<th>Debonding length $a$ 0–2 mm</th>
<th>Debonding length $a$ 2–27 mm</th>
<th>Debonding length $a$ 0–27 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>CL1FT180</td>
<td>$&gt;10,000,000$</td>
<td>$&gt;10,000,000$</td>
<td>$&gt;10,000,000$</td>
</tr>
<tr>
<td>CL5FT100</td>
<td>9,648,865</td>
<td>$*1,163,519$</td>
<td>$*10,812,384$</td>
</tr>
<tr>
<td>CL5FT120</td>
<td>1,389,837</td>
<td>201,405</td>
<td>1,591,242</td>
</tr>
<tr>
<td>CL5FT140</td>
<td>102,585</td>
<td>115,353</td>
<td>217,938</td>
</tr>
<tr>
<td>CL5FT160</td>
<td>54,611</td>
<td>146,918</td>
<td>201,529</td>
</tr>
<tr>
<td>CL5FT180</td>
<td>12,533</td>
<td>76,828</td>
<td>89,360</td>
</tr>
<tr>
<td>CL7FT100</td>
<td>608,864</td>
<td>865,175</td>
<td>1,474,038</td>
</tr>
<tr>
<td>CL7FT120</td>
<td>96,805</td>
<td>455,660</td>
<td>552,465</td>
</tr>
<tr>
<td>CL7FT140</td>
<td>223,823</td>
<td>367,879</td>
<td>591,701</td>
</tr>
<tr>
<td>CL7FT160</td>
<td>33,495</td>
<td>58,643</td>
<td>92,138</td>
</tr>
<tr>
<td>CL7FT180</td>
<td>9,898</td>
<td>38,050</td>
<td>47,948</td>
</tr>
</tbody>
</table>

*CL5FT100 listed the number of cycles at 15 mm due to the completion of the test.

Figure 3 shows the results of initial fatigue life (the number of cycles require for the debonding progress from 0-2 mm) in the function of nominal stress range. The black line (A–H) are $S$–$N$ curves of the general steel welded joint designed in guidelines called “Fatigue Design Recommendations for Steel Structures” by Japanese Society of Steel Construction (JSSC 1993). The blue line is the regression line resulted from the experiments. Result shows that it cross the D–H grade of fatigue strength grades of JSSC guidelines. Moreover, it can be seen that the more the nominal stress range increases, the more the fatigue strength grade decrease. Regarding to the fatigue limit, CL1FT180 case is out of the evaluation when organizing in nominal stress range, and the variation can be identified in the figure.

Figure 4, Figure 5 and Figure 6 present the $S$–$N$ curve evaluated by shear stress range, normal stress range and nominal stress range, respectively. In Figure 4, the fatigue limit of CL1FT180 cannot fully explain, and in Figure 5, the fatigue limit of CL1FT180 are fairly low. In contrast, in Figure 6, both the fatigue limit and the fatigue strength can be evaluated regardless to the number of layers of CFRP laminates. This is due to the consideration of nominal stress range which is the combination of shear stress and normal stress. From the result of the fatigue life evaluated by nominal stress range, the fatigue limit is considered to be approximately 15 MPa instead of 12.5
MPa (CL1FT180’s result). This value is very close to the result recalculated from (Lin et al. 2009) as well, in which the value of fatigue limit is 15.1 MPa.

Eq. 3 below refers to the equations and the correlation coefficients of each regression lines indicated in Figure 6, Figure 7 and Figure 8. It should be noted that the equations and the correlation coefficients were not included the result of the fatigue limit of CFRP laminates with 1 layer.

\[
\Delta \sigma_{sn} = 405.99N^{-0.091}, \ R = 0.90
\]

\[
\Delta \tau_{e} = 39.42N^{-0.094}, \ R = 0.92
\]

\[
\Delta \sigma_{e} = 60.18N^{-0.099}, \ R = 0.93
\]

\[
\Delta \sigma_{ep} = 77.88N^{-0.096}, \ R = 0.93
\]

Figure 7 shows the S–N curve of the fatigue strength at the debonding length of 0–2 mm and 0–27 mm. From the figure, when the nominal stress range increase, the initial fatigue life tends to decrease. Figure 8 shows the S–N curve of the debonding propagation life of adhesive in the range of 25 mm (debonding length of 2–27 mm). Eq. 4 below refers to the proposed equations of the fatigue strength (0–2 mm and 0–27 mm) and the debonding propagation life (2–27 mm) along with their correlation coefficients of each regression lines indicated in Figure 7 and Figure 8.

\[
\Delta \sigma_{ep, \ 0–2 \ mm} = 77.88N^{-0.096}, \ R = 0.93
\]

\[
\Delta \sigma_{ep, \ 0–27 \ mm} = 158.29N^{-0.146}, \ R = 0.93
\]

\[
\Delta \sigma_{ep, \ 2–27 \ mm} = 171.37N^{-0.100}, \ R = 0.83
\]
CONCLUSIONS

To sum up, the experimental results lead us to the following conclusions.

1. The fatigue life of adhesively bonded joints can be evaluated and its regression line equations can be proposed in the function of principle stress range independently to the thickness (number of layers) of the CFRP laminates.

2. The fatigue limit adhesively bonded joint is considered to be approximately at the principle stress range \( \Delta \sigma_{ep} = 15 \) MPa.

3. When the nominal stress is increased, the initial fatigue life tends to decrease.

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Japan Society of Civil Engineers (2013). “Advanced Technology of Joining for FRP Structures and FRP Bonding for Steel Structures”, Hybrid Structure Reports 09. [in Japanese]


EFFECTS OF SURFACE TREATMENT ON THE TENSILE AND BONDING PROPERTIES OF CARBON FIBERS

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ABSTRACT

In this article, the effects of electrochemical oxidation and sizing treatment of PAN-based carbon fibers (CFs) on the tensile properties, surface characteristics and bonding to epoxy resin were investigated. Electrochemical oxidation treatment of the carbon fibers was shown to improve the tensile strength by 9.8%, which is ascribed to the removed surface defects and impurities of the carbon fibers. Further sizing treatment exhibited a negligible effect on the tensile strength. Both oxidation and sizing treatments significantly improved the wettability and surface energies of the CFs by introducing oxygen-containing functional groups on the surface of the CFs. Microbond pull-out test was conducted to characterize the interfacial shear strength (IFSS) between a single carbon fiber and an epoxy droplet. As found, the oxidation treatment slightly increased IFSS from 71.24 MPa to 73.62 MPa. This is due to the formation of chemical bonds between resin matrix and carbon fibers, and the decrease of the mechanical interlocking. Further sizing treatment can significantly enhance the IFSS to 81.03 MPa due to the formation of vast chemical bonds. The effect of fiber treatment on the hydrothermal aging behavior of the bond between single carbon fibers and epoxy droplets was studied. It was found that the hydrothermal aging dramatically deteriorate IFSS of untreated fibers to resin matrix. However, the oxidation and sizing treatment can effectively improve the retention rate of IFSS.

KEYWORDS
Carbon fiber, bonding, interface, micro-bonding, hydrothermal ageing.

INTRODUCTION

Carbon fiber reinforced polymer (CFRP) composites are increasingly used in civil engineering because of their high specific strength/modulus and corrosion resistance. Durability of CFRP composites is one of the serious concerns when considering the long service lives of the civil engineering structures (generally more than 50 years). It is well known that the mechanical properties of CFRP composites are fundamentally determined by the tensile strength of the carbon fibers (CFs) and resin matrix as well as the interfacial bonding strength (Drzal and Madhukar 1993). Good interfacial adhesion allows more effective stress transferring from resin matrices to reinforcements, enhancing the ultimate strength. However, if the carbon fibers do not undergo any surface treatments after carbonization, the surfaces of CFs exhibit inertness (Zhang, Liu et al. 2013) in bonding and wetting to resin matrices. Furthermore, the interface between CFs with resin matrix, generally considered as the weakest region (Xu, Huang et al. 2007), tends to be damaged by the moisture or water ingress. Recent studies (Hong 2012, Li 2012) revealed that the epoxy resins showed insignificant degradation subjected to water immersion for a long time. However, remarkable deterioration of CFRP was observed, which is expected to be mainly coming from the damage of the interface.

In the present study, the effects of the electrochemical oxidation treatment and sizing treatment on the mechanical properties of single CFs and IFSS between fiber and epoxy resin were investigated. The study aimed to bring in an understanding of the effects of oxidation and sizing treatment on the mechanical properties of CFs, as well as the durability of the carbon fiber-resin bonding.

EXPERIMENTATION

Raw Materials
PAN-based CFs (800 tex) with diameter of 7 µm were investigated. The resin matrix used in this study was a commercial epoxy with a brand name of E51 (Xing-Chen Chemicals Co. Ltd., Wuxi, China), similar to EPON...
The curing agent was methyl tetrahydrophthalic anhydride (MeTHPA from Qing-yang Chemistry Co. Ltd., Jiaxing, China). The accelerator was a tertiary amine, tris (dimethylaminomethyl) phenol (DMP-30, from Shan-feng Chemical Industry Co. Ltd., Changzhou, China). Resin was mixed with the curing agent and accelerator at 100:80:2 by weight. Curing conditions were 3 h at 120°C, followed by 3 h at 150°C.

Characterization

The morphologies of untreated, oxidized and sized CFs were characterized with a Quanta 200F scanning electron microscope (SEM, USA). The chemical elements of the CF surfaces were detected using a Thermo Fisher Scientific K-Alpha X-ray photoelectric spectroscopy (XPS, USA) with an Al Kα X-ray (1486.6 eV) source.

Single carbon fiber tensile tests were carried out at ambient temperature with a crosshead speed of 0.00125 mm/s, according to ASTM D3379-75 using a JQ03A single fiber tensile tester (Zhongchen Digital Technic Apparatus Co. Ltd., Shanghai, China). The test gauge length was set as 20 mm. The diameters of each specimen were measured through an optic microscope before the test. At least 100 measurements were repeated for each type of CF. Tensile strength was analyzed with the two-parameter Weibull model.

The interfacial shear strength (IFSS) of CFs/epoxy composites was measured by the microbond pull-out test (Cui and Kessler 2012) with the equipment (FA-620, East Wing Industrial Co. Ltd., Japan) at a crosshead speed of 0.06 mm/min. The droplets of approximately 30–60 µm in diameter were selected to be tested. At least 50 measurements were conducted for each specimen.

For hygrothermal aging, three kinds of microbond samples were exposed to 95% RH chamber at 40°C. IFSS was measured periodically.

RESULTS

Tensile Properties of Single Carbon Fiber

As shown in Table 1, the tensile strength of the CFs is enhanced through electronic oxidation and sizing treatment. The shape parameter (m) and the scale parameter (σL) represent the variation degree and level of the tensile strength of CFs respectively. The higher the m value, the more stable the strength of the CF; the higher the σL value, the higher the strength of the CF. CFs treated by oxidation shows an increase in the shape parameter m from 5.85 to 7.47 and the scale parameter σL from 3.50 to 4.01. Meanwhile, further treatment of sizing leads to the increase of m to 7.90 and σL keeps almost constant. This means that oxidation brings in enhancement of the tensile strength and reduces the variance of the tensile strength of CFs. Furthermore, sizing treatment reduces the discreteness of the tensile strength, whereas the tensile strength is not enhanced anymore. After oxidation, both the tensile strength and the elongation at break are enhanced, while the modulus is unchanged. By further sizing treatment, only the tensile strength is enhanced. The diameter of the CFs is found to be reduced due to the electronic oxidation treatment, while the sizing treatment slightly increases the diameter. Similar results were found in (Liu, Tian et al. 2010).

![Figure 1 SEM photographs of CFs with various treatments. a) untreated fiber, b) oxidized fiber, and c) oxidized and sized fiber.](image_url)

The effect of the treatment on the surface morphology of CFs was analyzed with SEM. As shown in Figure 1a, SEM photograph of an untreated CF is rough and filled with grooves uniformly. There are some impurities and defects on the surface of the CF (Figure 1a). Oxidation seems to reduce the roughness (Figure 1b), and the grooves of the CFs become narrow and shallow. The impurities and defects on the surface of CFs are lessen and even
disappeared. This can be attributed to the fact that the oxidation reaction generally starts from the sharp edges of CFs and leads to the gentle and mild grooves and edges of CFs surfaces. Through oxidation, the micro-cracks of the surface of CFs are decreased and the stress concentration of the fiber is weakened (Liu, Tian et al. 2010). This can explain why the tensile strength was enhanced by electrochemical oxidation (see Table 1). The decrease in the modulus of CFs through electrochemical oxidation, as shown in Table 1, is due to the fact that the outside ordered sheath of the fiber is corroded by the electrolytic solution.

<table>
<thead>
<tr>
<th>Type</th>
<th>Diameter (μm)</th>
<th>Shape parameter m</th>
<th>Scale parameter σ0 (GPa)</th>
<th>Tensile strength (GPa)</th>
<th>Tensile modulus (GPa)</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Untreated</td>
<td>7.07± 0.34</td>
<td>5.85</td>
<td>3.50</td>
<td>3.24± 0.64</td>
<td>209.6± 8.6</td>
<td>1.58± 0.31</td>
</tr>
<tr>
<td>Oxidized</td>
<td>6.90± 0.37</td>
<td>7.47</td>
<td>4.01</td>
<td>3.76± 0.60</td>
<td>210.5± 10.4</td>
<td>1.79± 0.30</td>
</tr>
<tr>
<td>Oxidized + Sized</td>
<td>6.93± 0.34</td>
<td>7.90</td>
<td>3.98</td>
<td>3.75± 0.56</td>
<td>210.2± 9.2</td>
<td>1.78± 0.33</td>
</tr>
</tbody>
</table>

Further sizing produces flat and smooth CFs surface (Figure 1c). It is believed that the grooves on the CF surface are filled with the sizing agent. As believed, the sizing agent cannot change the surface structure of CFs (the ordered outside sheath), and thus it will not vary the mechanical properties of CFs as indicated in Table 1.

Interfacial shear strength between single fiber and epoxy droplet

Figure 2 shows the IFSS between three kinds of CFs and epoxy resin. Compared to the untreated fibers, oxidation of CF increases the IFSS from 71.24 MPa to 73.62 MPa. Further sizing treatment brings in a remarkable increase in IFSS to 81.03MPa. Clearly, the combination of the electrochemical oxidation and sizing significantly improves the interfacial bonding of the CF to epoxy resin, while only electrochemical oxidation does not.

The improvement in the bonding strength for the oxidized and sized CFs to epoxy resin is due to the increased active functional groups on the surface of the CFs. This can be verified by the presented chemical groups on the fiber surface detected by XPS. Compared with C-C functional groups, the C-O-C/C-OH functional groups tend to form stable bonds with the epoxy resin. Yuan et al. (Yuan, Zhang et al. 2013) indicated that the increased wettability and possibility of forming chemical bonding could account for the enhanced interfacial shear strength between the sized CF and epoxy resin. The increased surface energy of the sized CFs makes the fibers to be easily wetted by the resin solution, which increases the possibility of the molecular contact between the epoxy and the sized CFs. This has been proved by the contact angles between epoxy droplets and CFs, and surface energies of the CFs (Table 2). The interface region is formed by the penetration and curing of the sizing resin system with the surrounding resin matrix. In addition, the sizing agent is mainly composed of epoxy resin. According to the “similar dissolve mutually theory”, the chemical reactions between carbon surface, sizing agent, and E-51 resin easily occur (Viswanathan, Wang et al. 2001, Liu, Ge et al. 2012). The COOH functional groups existing on the CF surface may react with C-OH of the sizing agent and creates a strong bond. Thus, the good adhesion is obtained between the matrix and the sized CFs.

Figure 2 IFSS between E51-MeTHPA resin and CFs.
The results of XPS for different CFs are listed in Table 2. For untreated fibers, the carbon concentration on the surface is only 82.40% while the oxygen concentration is as high as 16.16%. Through electrochemical oxidation and sizing treatment, the oxygen concentration initially decreases from 16.16% to 13.36%, and then increases to 17.04%. This anomalous result of oxygen concentration through electrochemical oxidation is greatly different from literature (Cao, Huang et al. 2005, Liu, Tian et al. 2010). As believed, this may be because the untreated CFs obtained from the export of the high temperature charcoal stove carry much detached oxygen concentration from air. Through electrochemical oxidation, the detached oxygen concentration is firstly removed by the ammonium bicarbonate solution and then the carbon concentrated on the surface of fibers is oxy-generated to oxygen function groups. This assumption is validated by the subsequent analysis of functional groups on the CF surfaces. After electrochemical oxidation, the nitrogen concentration increases significantly from 1.43% to 2.21%, in agreement with a previous study (Cao, Huang et al. 2005). This indicates that the nitrogenous functional groups were created on the fiber surface by the reaction with ammonia ions in the electrolyte. Compared with the untreated CFs, oxidized fibers show a slight increase in the concentration of C-OH/C-O-C from 11.0% to 14.8%, the concentration of C=O from 5.4% to 8.3% and the concentration of COOH from 3.3% to 4.9%. The increase of oxygen-containing functional groups on the surfaces may be due to the fact that a certain number of C=O functional groups on the CF surface is reacted with the oxygen into C-OH/C-O-C, and then further oxidized into C=O and COOH during the electrochemical oxidation process. After sizing treatment, the concentration of C-OH/C-O-C increases dramatically from 14.8% to 40.7%, while the C=O and COOH functional groups disappear. The change is owing to the fact that the main component of the sizing agent, mainly composed of epoxy resin, contains a large amount of C=O active functional groups. The introduction of the active functional groups significantly improves the surface activity, polarity, and hydrophilicity of the CFs, resulting in improved wettability of CFs. At the same time, the increase of C-OH/C=O-C, C=O and COOH functional groups, which contain activated carbon atoms, can form stable chemical covalent bonds with the C-O-C functional groups in epoxy resin. This is believed to enhance the IFSS of CFs effectively.

Table 2 Percentages of elements and several oxygen containing groups on the surfaces of CFs

<table>
<thead>
<tr>
<th>Type</th>
<th>NIs/Cls</th>
<th>C-C</th>
<th>C-OH</th>
<th>C=O</th>
<th>COOH</th>
</tr>
</thead>
<tbody>
<tr>
<td>Untreated</td>
<td>0.0174</td>
<td>80.3</td>
<td>284.7</td>
<td>11.0</td>
<td>286.1</td>
</tr>
<tr>
<td>Oxidized</td>
<td>0.0256</td>
<td>72.0</td>
<td>284.3</td>
<td>14.8</td>
<td>285.6</td>
</tr>
<tr>
<td>Oxidized + Sized</td>
<td>0.0148</td>
<td>59.3</td>
<td>284.4</td>
<td>40.7</td>
<td>286.0</td>
</tr>
</tbody>
</table>

For electrochemical oxidized carbon fibers, IFSS due to mechanical interlocking is decreased, while the portion of IFSS due to chemical bonds is increased (Table 2). Consequently, IFSS between oxidized CFs and epoxy is not enhanced remarkably (Figure 2). The CFs treated by oxidation and sizing possess a lot of C-O-C/C-OH active functional groups (Table 2). As a result, the chemical bonds are remarkably increased, leading to a significant increase in IFSS.

Figure 3 The effect of the aging on IFSS between E51-MeTHPA resin and CFs.
Furthermore, IFSS after hydrothermal aging was also investigated for three kinds of CFs based on microbond pull-out test, as shown in Figure 3. The decreases in IFSS are from 71.24 MPa to 42.29 MPa for untreated fiber, from 73.62 to 50.32 MPa for oxidized fiber, and from 81.03 MPa to 56.43 MPa for sized fiber, respectively. The retained percentages of the IFSS are 59.4%, 68.4% and 69.6% for the untreated, oxidized, and sized, respectively when the samples are aged for 24 days. Clearly, the bonding between the carbon fibers and epoxy resin are very susceptible to the hydrothermal exposure. The oxidation and sizing treatments enhance the resistance to the hygrothermal aging, but to a limited extent, especially for a long period.

CONCLUSIONS

Effects of electrochemical oxidation and sizing treatments on the tensile properties, wettability and interfacial shear strength to epoxy resin of PAN-based CFs was investigated in the present study. The following conclusions were drawn based on the testing results.

Electrochemical oxidation treatment can improve the tensile strength of CFs by 9.8% through reducing the roughness and removing the surface defect and impurity. Further surface sizing treatment only smooth the surface of CFs through filling the grooves and has a negligible effect on the tensile strength.

Combining electrochemical oxidation with surface sizing treatments can tremendously improve IFSS of CF/EP and the resistance to hydrothermal aging of the interfacial bonding. This is attributed to the formation of effective bonding to epoxy resin, due to the oxygen-containing functional groups on the carbon fibers introduced by the treatments.

REFERENCES


BOND STRENGTH DURABILITY OF BFRP BARS IN CONCRETE SUBJECTED TO ELEVATED TEMPERATURE AND ALKALINE ENVIRONMENT

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ABSTRACT

The use of basalt fiber–reinforced polymer (BFRP) bars as a reinforcing material for concrete structures has gained increasing interest worldwide. Limited studies have been reported on the performance of BFRP bars in concrete when exposed to harsh environments. This paper presents the test results of an experimental study aimed at investigating the bond strength durability of BFRP bars embedded in concrete when exposed to accelerated environmental effects. The experimental program included testing and investigating BFRP deformed bars with 12-mm diameter. Pullout specimens were tested under direct tensile load after being exposed to an alkaline solution (pH=12.9) for up to 6 months at elevated temperatures of 40°C. Effects of alkali environment and exposure periods on the bond strength, degradation mechanism, and mode of failure of reinforced specimens with BFRP were investigated. The test results revealed that the average bond strength of conditioned specimens after 1.5, 3, and 6 months of exposure at 40°C were 14.8, 13.3, and 12.5 MPa, while the bond strength of unconditioned specimens was 15.5 MPa. Accelerated alkaline environments had a harmful effect on bond strength of conditioned specimens over time. After 6 months of exposure, the bond strength retention was 81% compared to that of the unconditioned specimens.

KEYWORDS

Bond, Basalt-fibres, Concrete, Durability, Pullout, BFRP, Bars.

INTRODUCTION

The use of basalt fiber–reinforced polymer (BFRP) bars in construction applications is relatively new. This type of structural material is expected to provide a performance comparable or superior to that of glass-fiber–reinforced polymers (GFRP), while being significantly cost-effective [Wang et al. (2012); ElSafty et al. (2014)]. Numerous research efforts have been conducted on the bond durability of GFRP and carbon (FRP) bars in concrete [Bakis et al (2007); Davalos et al. (2008); Robert and Benmokrane (2010)]. However, very limited studies have investigated the bond durability of BFRP bars embedded in concrete when subjected to harsh environment conditions. Before the use of BFRP as reinforcement for concrete structures and inclusion in FRP standards and guides, more experimental investigations on BFRP bars are needed including mechanical properties, structural and mechanical behaviors, durability, and bond strength to concrete. It is worth mentioning that BFRP bars have not yet been incorporated into design standards and specifications. Therefore, the current paper aims at narrowing the gap in our knowledge on BFRP performance. The work presented herein is a part of a large research study on the use of BFRP bars as reinforcement. This study provides an insight into the bond of BFRP bars with concrete after long-term environmental conditioning. The findings of this work contribute to integrating BFRP bars into FRP standards and guides, such as [ACI 440.6M (2008), CSA S807 (2015), and CSA S806 (2012)].

EXPERIMENTAL WORK

Material Properties

This study investigates a 12-mm-diameter BFRP bar (nominal cross-sectional area of 125 mm²) with a deformed surface. Figure 1 shows the surface configuration of the investigated BFRP bar. The BFRP bar was made of continuous longitudinal basalt fibers bound together with vinylester resin using a pultrusion process. The basalt
fibers were produced from volcanic material with organic surface coating (ASA.TEC, Austria). Table 1 summarizes the physical and mechanical properties of the investigated BFRP bar. As seen in Table 1 the preliminary tests showed that the newly developed 12-mm BFRP bar satisfied the minimum requirements of ACI 440.6M (2008) and CSA S807 (2015) concerning their physical and mechanical properties for use as non-prestressed reinforcement for concrete structures. Different laboratory tests on the investigated bar can be found in Hassan and Benmokrane (2015). All pullout specimens were cast using a ready-mixed, normal-weight concrete with a 28-day target concrete compressive strength of 30 MPa. The concrete mixture consisted of 350 kg of Type 10 cement (corresponding to ASTM I cement), 813 kg of fine aggregate, 1032 kg of coarse aggregate, and 154 kg of water per cubic meter of concrete. The concrete compressive strength ($f'_c$), determined by testing six 100×200 mm concrete cylinders, ranged from 31 to 34 MPa. After one week of curing, all specimens and cylinders were left outside the laboratory for 4 weeks before being conditioned.

![Figure 1 BFRP bar with 12-mm diameter](image)

Table 1 Physical and mechanical properties of 12 mm- control BFRP bars

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Physical</td>
<td>Cross-sectional area (mm$^2$)</td>
<td>125</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>Fiber content by weight (%)</td>
<td>8.15</td>
<td>55% (by vol.)</td>
<td>70 (by wt)</td>
</tr>
<tr>
<td></td>
<td>Density (gm/cm$^3$)</td>
<td>2.09±0.02</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>Moisture uptake (%)</td>
<td>0.13±0.006</td>
<td>&lt; 1</td>
<td>1.0 (D2); 0.75 (D1)</td>
</tr>
<tr>
<td></td>
<td>Tg (ºC)</td>
<td>117.0±2.65</td>
<td>100</td>
<td>80 (D2); 100(D1)</td>
</tr>
<tr>
<td>Mechanical</td>
<td>Ultimate tensile strength ($f_u$) (MPa)</td>
<td>1706±40</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Modulus of elasticity ($E_f$)(GPa)</td>
<td>62.1±5.6</td>
<td>39.3 GPa</td>
<td>40.0 GPa</td>
</tr>
<tr>
<td></td>
<td>Ultimate tensile strain ($\varepsilon_u$) (%)</td>
<td>2.52±0.2</td>
<td>&gt; 1.2%</td>
<td>&gt; 1.2%</td>
</tr>
<tr>
<td></td>
<td>Bond strength ($\tau_{max}$) (MPa)</td>
<td>15.48±0.4</td>
<td>&gt; 9.6 MPa</td>
<td>&gt; 8 MPa</td>
</tr>
</tbody>
</table>

*D1 and D2 classifications can be found elsewhere in (CSA 2015).

* The mechanical properties are calculated using the nominal cross-sectional area.

**Test Specimens and Procedures**

The pullout specimens were comprised of a 1200-mm long BFRP bar embedded centrally inside a concrete prism dimensions (200 × 200 × 200 mm). The embedded length was kept constant at 5$d_b$, where $d_b$ is the nominal diameter of BFRP bar. Plastic tubes were used at the loading end of the bar to minimize the stress concentration near the loading plate. Steel pipes were used as anchors and were cast with cement grout before testing. Figure 2 depicts typical specimen details. A total of 20 specimens were tested under direct pullout tests. Fifteen specimens were fully immersed in an alkaline solution at elevated temperatures of 40ºC for 1.5, 3.0, and 6 months while five unconditioned specimens were tested at room temperature for comparisons. All specimens were tested under pullout testing according to ACI 440.3R (2004) test method B.3. The tests were carried out using a Baldwin testing machine at displacement control mode with a maximum rate of 20 kN/min. The slip at the free-end of the BFRP bar was measured using a linear variable displacement transducer (LVDT). Figure 2 shows the test setup and instrumentation of the test specimens. The applied load and bar slippage were recorded automatically throughout the test with a data-acquisition system. The maximum bond stress at the peak load, $\tau_{max}$, was calculated assuming a uniform bond stress distribution along the embedded length of the bar in concrete, $l_d$, using Eqn. (1).

$$\tau_{max} = \frac{P_{max}}{\pi d_b l_d}$$

where $\tau_{max}$ is the maximum bond stress (MPa), $P_{max}$ is the peak load during pullout (kN), $d_b$ is bar nominal diameter (12 mm), and $l_d$ is embedded length in the concrete (=60 mm).

**Environmental Conditioning**

The pullout blocks were fully immersed in alkaline solution inside big steel tanks while the free length of BFRP bars remained freely unconditioned inside the environmental chambers. The alkaline solution compositions consisted of 118.5 g Ca (OH)$_2$ (calcium hydroxide), 4.2 g KOH (potassium hydroxide) and 0.9 g NaOH (sodium...
hydroxide) in 1 L of deionized water according to ACI 440.3R (2004). The alkaline solution pH value was 12.9. The pullout specimens were separated from each other and from the bottom of tank to allow the solution to circulate freely between and around the blocks. The top of the steel tank was hermetically sealed with a polyethylene film to prevent excessive evaporation of water during the conditioning. In addition, the water level was kept constant throughout the study to avoid the pH from increasing as a result of the water evaporation. In order to accelerate the BFRP bar-concrete bond degradation, the test specimens were placed at elevated temperatures of 40°C for exposure periods of up to 6 months. Figure 3 shows the test specimens inside the conditioning tank in the environmental chambers.

![Figure 2 Test specimen geometry details and test setup](image)

**Figure 2 Test specimen geometry details and test setup**

![Figure 3 Test specimens in the conditioning tank inside the environmental chamber](image)

**Figure 3 Test specimens in the conditioning tank inside the environmental chamber**

**TEST RESULTS AND DISCUSSION**

**Mode of Failure**

All conditioned BFRP-reinforced specimens failed in a typical pullout mode of failure by exhibiting slip through the free end with no signs of splitting cracks appeared in the cube specimen. Similar behavior was observed for both unconditioned and conditioned tested specimens. Figure 4 shows typical mode of failure of the tested specimens. Some prisms were split after testing for a closer inspection of the concrete surface at the location of embedded length. The visual examination at the location of embedded length showed some tiny chopped basalt fiber from the coating surface of the bar attached to the concrete surface. On the other hand, examination of BFRP bar surface showed residues of concrete between the deformations, after pullout, and minor scratches at some rib
locations (Fig. 4). The final mode of failure of the tested specimens was dominated by shearing of the concrete located at the BFRP bar-concrete interface. However, an abrasive type fracture in either the concrete or the composite bar itself could be involved.

![Figure 4 Typical pure pullout failure](image)

**Table 2** Test matrix and bond test results for basalt pullout-tested specimens

<table>
<thead>
<tr>
<th>Immersion time (Months)</th>
<th>Temperature (°C)</th>
<th>Peak load (kN)</th>
<th>$\tau_{\text{max}}$ (MPa)</th>
<th>$\tau_{\text{aver.}}$ (MPa)</th>
<th>COV (%)</th>
<th>Retention (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconditioned</td>
<td>23</td>
<td>34.4</td>
<td>15.2</td>
<td>15.5±0.4</td>
<td>2.9</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>36.5</td>
<td>16.1</td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td></td>
<td>35.6</td>
<td>15.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>34.6</td>
<td>15.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>34.1</td>
<td>15.1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>40</td>
<td>30.7</td>
<td>13.6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>36.6</td>
<td>16.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>37.3</td>
<td>16.5</td>
<td>14.8±1.5</td>
<td>10.2</td>
<td>96</td>
</tr>
<tr>
<td></td>
<td></td>
<td>33.3</td>
<td>14.7</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td>29.5</td>
<td>13.0</td>
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<tr>
<td>3</td>
<td>40</td>
<td>24.1</td>
<td>10.6</td>
<td></td>
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</tr>
<tr>
<td></td>
<td></td>
<td>41.8</td>
<td>18.5</td>
<td></td>
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<tr>
<td></td>
<td></td>
<td>27.6</td>
<td>12.3</td>
<td>13.3±3.1</td>
<td>23.1</td>
<td>86</td>
</tr>
<tr>
<td></td>
<td></td>
<td>30.5</td>
<td>13.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>26.3</td>
<td>11.6</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>6</td>
<td>40</td>
<td>26.1</td>
<td>11.6</td>
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<td>35.6</td>
<td>15.7</td>
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<td>26.4</td>
<td>11.7</td>
<td>12.5±1.9</td>
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<td>81</td>
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<td>25.0</td>
<td>11.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>28.1</td>
<td>12.4</td>
<td></td>
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</tr>
</tbody>
</table>

Note: In all cases, the mode of failure was pullout; $\tau_{\text{max}}$ = maximum bond stress at the peak load (MPa).

**Bond strength and Stress-Slip Responses**

Figure 5 shows the bond stress and free-end slip relationships for all tested specimens. The free-end slip was recorded directly from the LVDT at the unloaded end. The bond stress vs. free-end slip curves showed similar trends for the unconditioned and conditioned pullout specimens; however, the bond strengths after environmental conditioning were different. The free-end slip remained zero until the bond stress reached high levels, compared with the ultimate bond stress. Thereafter, a pre-peak response was observed up to their peak bond stress then followed by a post-peak response up to 87% (on average) of their peak stress. Finally, a sudden loss of bond strength accompanied with a sudden free-slip was observed. The average bond strength (based on an average of five test results) was calculated as the pullout force over the embedded area of the bar (Eqn.1). The peak bond load and corresponding maximum bond stress of all unconditioned and conditioned specimens are reported in Table 2. As shown in Table 2, the average bond strengths of the conditioned specimens after 1.5, 3, and 6 months of alkaline environmental exposure at 40 °C were 14.8, 13.3, and 12.5 MPa, respectively, while the bond strength of unconditioned specimens was 15.5 MPa. This indicates that the bond strength of the BFRP bar-concrete interface decreased as exposure time increased. The bond strength retention after 6 months of exposure was 81%. It should be noted that the bond strength after 6 months of exposure at 40 °C was 12.5 MPa, which is higher than the specified limits for bond strength in ACI 440.6M (2008) (> 9.6 MPa) and CSA S807 (2010) (> 8 MPa).
Effect of Exposure Period on Bond Degradation

Figure 6 shows the effect of exposure period on the bond strength and strength retentions of the BFRP bars at 40°C. As seen, the bond strength trend decreased as the time of immersion increased. The suggested bond deterioration mechanism of tested specimens is described. After the environmental conditioning, gaps along the interface between the bar and concrete could be created due to the variation in coefficient of thermal expansion (CTE) of both materials which imposes radial bursting force on the concrete surface at the interface. When the developed stress in concrete is larger than its tensile strength, cracks will develop. Thus, FRP bar and concrete will not be in contact as tightly as before and thus the bond strength is decreased [Belarbi and Wang (2012)]. Furthermore, when immersed in alkaline solution, chemical ions filled these voids, which facilitate the attack of FRP bar’s polymeric matrix, especially in the outer surface and results in degradation of reinforcement itself and may contribute to the bond strength degradation [Abbasi and Hogg (2005), Chen et al. (2007), Belarbi and Wang (2012) and El Refai el al. (2014)].

CONCLUSIONS

The main objective of this study is to investigate the bond durability of BFRP bars embedded in concrete when exposed to harsh environments. A bond durability investigation was performed on BFRP-reinforced pullout specimens that were fully immersed in an alkaline solution and subjected to elevated temperatures. Based on the test results and discussions, the following conclusions can be drawn:
1. All BFRP-reinforced specimens failed in a typical pullout mode failure by exhibiting slip through the free-end. The final mode of failure was dominated by shearing of the concrete located at the BFRP bar-concrete interface.

2. The average bond strength of conditioned specimens after 1.5, 3, and 6 months of exposure at 40 °C were 14.8, 13.3, and 12.5 MPa while the bond strength of unconditioned specimens was 15.5 MPa. The bond strength results after 6 months of exposure fulfilled the minimum requirements of the bond strength for use as non-prestressed reinforcements for concrete structures according to the ACI and CSA specifications.

3. Accelerated alkaline environments had a detrimental effect on bond strength of conditioned specimens’ over time during its immersion. After 6 months of exposure, the loss in the bond strength was 19% in comparison to their counterpart unconditioned specimens.

ACKNOWLEDGMENTS

The authors would like to acknowledge the support of the Natural Sciences and Engineering Research Council of Canada (NSERC), the Fonds québécois de la recherche sur la nature et les technologies (FQRNT), ASA.TEC the basalt FRP bar manufacturer, Florida Department of Transportation (FDOT), and the technical staff at the Department of Civil Engineering at University of Sherbrooke.

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MODELING OF THE EFFECT OF WATER INGRESS ON THE BOND BETWEEN CARBON FIBER – EPOXY RESIN WITH THE MOLECULAR DYNAMICS TECHNIQUE

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ABSTRACT

FRP (fiber reinforced polymer) has taken advantages of the fiber reinforcement and resin matrix, possessing excellent mechanical properties and corrosive resistances, even in harsh environments. The lack of the knowledge of the durability performances of FRPs in various environmental conditions and the related degradation mechanisms of FRPs leads to significantly conservative design. Water ingress plays a critical effect on the durability of FRP, due to susceptibility of the bond between the fibers and resin matrices. In the present paper, molecular dynamics technique was used to model the moisture effect on the fiber-resin bond. The bonding performances of the epoxy on carbon fibers under dry and wet conditions were simulated. Free energy profiles were calculated to quantify interfacial adhesion. As shown from the decline of binding free energy, the water ingress led to remarkable deterioration of the fiber bonding to the resin matrix. The study is expected to be helpful for both the development of durable FRPs, and to predict the service life of a FRP subjected to hydrothermal aging accurately.

KEYWORDS

FRP, interface, water, molecular dynamics.

INTRODUCTION

FRP (fiber reinforced polymer) has been widely applied in many areas due to its lots of advantages, e.g., high specific strength/modulus, easily production, etc (Karabinis and Rousakis 2002, Nystrom, Watkins et al. 2003, Sen 2003). The service environments of FRPs such like marine may lead to deterioration of FRPs. Moreover, some application of FRP requires the material to serve for over 50 years (for example, submarines and bridges). The long-term durability of FRP is always a concern due to their broad applications. Indeed, design with respect to long-term durability has been largely based on experience(Davies and Rajapakse 2014), and safety factors are often too conservative compared to the field data. This is related to the lack of the knowledge of the durability performances in various environmental conditions, and the related degradation mechanisms. The interfaces between fibres and resin matrix were found to play a critical effect on the durability of FRP, especially in case of water ingress(Davies and Rajapakse 2014, Wang, Huang et al. 2015).

There are many experimental techniques to test the bond strength between the carbon fiber(CF)/epoxy interface, such as the fiber pull-out and the fiber push-out test. Those technologies are widely applied. However, CF/epoxy interface is intrinsically inhomogeneous and anisotropic. In addition, the bonding structure is asymmetry and the entropic constraints are provided at a phase boundary(Mansfield and Theodorou 1991). These three factors lead to the complex of the fiber-resin interface, thus make it hard to establish clear physically based models for the coupling between water ingress and mechanical behaviour concerning long-term performance(Davies and Rajapakse 2014).

Molecular simulation approaches can complement experiments by providing a detailed study of the interaction in composites(Rissanou and Harmandaris 2014). Three hundred years ago, molecular dynamics are used to study the property of interface(Dholakia, Chandra et al. 1731). Numerous molecular simulations have been reported to studied the various property of interfacial ever since, e.g., tensile strength, surface free energy, adhesive tension, etc(Dholakia, Chandra et al. 1731, Broughton and Gilmer 1983, Mansfield and Theodorou 1991, van Buuren, Marrink et al. 1993, Pasquarello, Hybertsen et al. 1998, Bizzarri and Cannistraro 2002, Kuo and Mundy 2004, Nagayama and Cheng 2004, Rissanou and Harmandaris 2014). For example, the interfaces of epoxy/concrete and
epoxy/wood have been studied using molecular simulation method in the past decade. The adhesion was estimated by pulling the epoxy molecules off the surface of silica and cellulose under dry and moisture condition. Take XG/clay for another example, the adhesion was calculated by pulling the polymer off the clay surface (Wang, Wohlert et al. 2014). The surface energy was calculated and it is proven water effectively deteriorate the interface adhesion in those interfaces. However, molecular simulations of FRP mainly focus on the properties of carbon fibers and epoxy separately, or on the interfacial adhesion between FRP and other material. Few studies have focused on the molecular simulation of CF/epoxy interfacial adhesion in FRP.

Understanding CF/epoxy interfacial behaviour at molecular level is important to judiciously select and efficiently design FRP. Molecular dynamics (MD) simulation are helpful as could provide insights into the fiber-resin bonding mechanisms, and reveals the effect of water ingress on the bonding (Haile 1992).

In the present paper, an attempt was performed to investigate the moisture effect on the fiber-polymer bonds in FRP with experimental technique and molecular dynamics technique. The FRP with carbon fiber and epoxy resin was selected which is used extensively in civil engineering, e.g., structural rehabilitation. In the current work, there is an multi-scale analysis of the water ingress effects on FRP interfacial bonding propriety.

A microbonding pull-out test (Miller, Muri et al. 1987) was performed to test the interfacial shear strength (IFSS) of the epoxy resin (DGEBA, MeHHPA and DMP30 system) to a single fiber. A short overview of the simulations and the model systems is suggested in Section 3. The adhesion between E51 epoxy and carbon fiber in our simplified model was simulated using Molecular Dynamics (MD) methods. Section 4 compares the experimental and simulated results.

**EXPERIMENTAL**

The interfacial shear strength (IFSS) of a single carbon fiber to an E51 epoxy resin was measured by the microbonding pull-out test (Qian Zhen Ming Ming 1996) with the equipment (FA-620, East Wing Industrial Co., Ltd., Japan) at a crosshead speed of 0.06 mm/min.

The resin droplets of approximately 15 µm to 20 µm in diameter were selected to test. The epoxy resin was DGEBA mixed with MHHPA and DMP30 at ratio of 100:80:2 by weight. TC36S carbon fiber (made by TaiLi Industrial Co., Ltd., Taiwan, China) in the test (Wang, Huang et al. 2015). The droplets of E51 epoxy on the fibers were cured at 120 °C for 1 hour and 150 °C for 1 hour in a hot oven.

Fifty E51/TC36S carbon fiber samples were made to study the water coupling between water ingress and mechanical behaviour concerning long-term performance in FRP. They were respectively treated 7 days, 30 days, 60 days, 90 days immersed in water, ten for each.

**SIMULATION WITH MOLECULAR DYNAMICS**

Material Studio software was used to build a carbon fiber-epoxy resin interface model. The forcefield used here is Consistant Valence Forcefield (CVFF)(Dauber - Osguthorpe, Roberts et al. 1988). The applicability of CVFF in the investigation of FRP mechanical property has been validated in many other studies (Zhou, Tam et al. 2015). The interface model consist of a simplified E51 epoxy model and a simplified carbon fiber model.
**E51 EPoxy Model**

In the epoxy model, Diglycidyl Ether of Bisphenol A (DGEBA), hexahydro-4-methylphthalic anhydride (MeHHPA) was used as the base composition. The curing agents were 2,4,6-tris-(dimethylaminomethyl) phenol (DMP-30). Figure 2 depicts the chemical reaction of the resin system.

![Chemical Reaction](image)

**Figure 2** Curing reaction of the resin system.

As shown in Figure 2, each DGEBA can is connected to another molecule through its opposite epoxide head. Figure 3(a) shows the polymer structure of DGEBA/MeHHPA cross-linkage. There are three main steps in our cross linking process, as follow:

**Construction and Pre-equilibration**

One amorphous DGEBA/MeHHPA cell with an initial density of 1.2g/cm³ was created under periodic boundary conditions at room temperature. The cell is composed of 16 DGEBA and 36 MeHHPA. Then the system was exposed to 1000 steps minimization to reach the minimum energy. Next the amorphous cells is equilibrated for 10 ps in the canonical ensemble (NVT) dynamics at 298K, with the integration time step of 1 fs. Another isothermal-isochoric ensemble (NPT) dynamics at 298 K and 1 atm last for 10ps were performed on the amorphous cells to reach an equilibrium.

![Atomic Model](image)

**Figure 3** Atomic model

(a) E51 epoxy  (b) Carbon fiber

(c) the CF/epoxy interfacial model in dry condition  (c) the CF/epoxy interfacial model in water ingress condition
**Creation of covalent bonds**

Once does each structure reach an equilibrium condition, the distances between pairs of reactive atoms are measured. Covalent bonds will be formed between suitable atoms (those within cross linking cut-off distance, here 4Å-10Å were chosen). In order to relieve the stress forced in the system, a smart minimization task with 1000 iterations was performed. After that, the system was exposed to 10 ps high temperature canonical (NVT) dynamics at the temperature of 500K with each step of 1fs. This process gives enough kinetic energy to the molecules and enhances the possibility of reacting sites to be posed in the reaction cut-off distance. This cross-linking process has been repeated several times until no reactive atoms are identified.

**Post-equilibration**

After the cross-linking process is completed, the cell is treated with a relaxation process by exposed to 10 ps temperature canonical (NVT) dynamics at the temperature of 500K, 430K, 360K and 298K. Here an epoxy model with 83.56% cross-link containing 1573 atoms in the box with the dimension of 27×27×27 Å³ (see Figure 3.a) were obtained.

**INTERFACIAL MODEL**

Carbon fiber cell was simplified to be several paralleled graphene sheets with the initial density of 1.76g/cm³ at the dimensions of 89×90×15 Å³ (see Figure 3.b). To start the simulations, the epoxy was placed paralleled to the carbon fiber surface (see in Figure 3). The epoxy and carbon fiber were put in a periodical orthorhombic box with the same lateral dimensions as the fiber sheet, making the sheet effectively infinite in size. Then the simulation configurations were relaxed by sequentially performing 100ps canonical (NVT) at 298K, 100ps isothermal-isochoric ensemble (NPT) dynamics at 298 K and 1atm, 100ps canonical (NVT) at 298K. After the process, the E51 epoxy molecules were spontaneously adsorbed to the carbon fiber surface (see in Figure 4(a)). This molecular dynamics simulation was performed using LAMMPS simulation package, with a basic time step of 1fs, and with forcefield CVFF.

To assess the molecular adhesion of the epoxy to carbon fiber, the potential of mean force (PMF) pertaining to the desorption process was calculated by Metadynamics (Laio and Gervasio 2008) using PLUMED plug-in package (Bonomi, Branduardi et al. 2009). In the metadynamics simulation, a Gaussian hill energy was added to the system every 100 steps, with a basic time-step of 1fs, with the height of 0.05KJ/mol, and the width of 0.35.

For a compound material, interfacial adhesion in the presence of water is critical. Thereafter, water molecules were put into this interfacial system to study the water effects on the interface bonding (see in Figure 4(b)). Water molecules were represented using the simple point charge (SPC) model of Berendsen (Berendsen, Postma et al. 1981). The simulation configurations were relaxed by sequentially performing 100ps canonical (NVT) at 298K, 100ps isothermal-isochoric ensemble (NPT) dynamics at 298 K and 1atm, 100ps canonical (NVT) at 298K. After the process, the E51 epoxy molecules still adsorbed to the carbon fiber surface (see in Figure 4(b)). Next, the same rule of Metadynamics simulation was performed on this interfacial system.

![Figure 4: The interface model](image-url)
RESULTS AND DISCUSSIONS

The results of the experiment and simulation are presented in this section, mainly focus on the effect of water in the bonding performance of FRP material.

Interfacial shear strength of E51 epoxy/ carbon fiber

Figure 5 presents the experimental results of the interfacial shear strength (IFSS) between the single carbon fiber and epoxy droplet with the microbonding pull-out method. These E51 epoxy/carbon fiber samples were immersed in water for 0, 7, 30, 60 and 90 days, respectively. As shown, a remarkably decrease in the interfacial shear strength of E51 epoxy/carbon fiber under moisture condition can be found. The IFSS of the E51 epoxy/ carbon fiber sample under dry condition is 139Mpa, which is 1.98 times of the sample under the condition of 90 days immersed in water, whose IFSS reduced to 70Mpa. The IFSS of the samples decreased rapidly after 7 days immersion in water. Then with the increase of immersion days from 7 days to 90 days, the IFSS decrease still remarkable, but slowly.

![Figure 5 The interfacial shear strength of E51 epoxy/carbon fiber as a function of water immersion days.](image)

Small molecules of water penetrate into the interface by diffusion and filtering through voids and cracks of the resin or by capillary migration along the fibers. The water then forms a weak hydrogen bond with E51 epoxy molecules (Wang, Huang et al. 2015).

In order to clarify the effect of water in the bonding performances of the epoxy/carbon fiber from microscopic scale, MD simulations are performed and the influence of water molecules are illustrated in the next section.

Molecular Adhesion

Here the molecular adhesion was simulated by atomic model in order to enlighten the effect of water on the interfacial bonding. The E51 epoxy/Carbon fiber interface model under the dry condition and wet condition were built respectively. The later condition is achieved by adding a number of water molecules into the model. As shown in Figure 4, after the system reached the equilibrium, under the dry condition, the E51-epoxy molecule is attached to the surface of carbon fibers, the distance between the center mass of epoxy molecules and the surface of carbon fibers in vertical direction is around 5 Å. In the wet condition, however, the distance between the center mass of E51 epoxy and the surface of carbon fibers in vertical direction is around 6.6 Å. The epoxy molecules are slightly detached from the fiber surface as the water molecules entered the area between E51 epoxy molecules and carbon fiber molecules and formed hydrogen bonds with E51 epoxy molecules (it has been calculated that some water molecules is within the distance of 3.5 Å from E51 epoxy molecules).

In the detachment simulation using metadynamics method, E51 epoxy molecules were pulled away from the surface of carbon fibers, the potential of mean force (PMF) of the E51 epoxy/ Carbon fiber interface in the process were calculated, and the free energy profiles of metadynamics simulation of the interface in dry and wet condition are constructed and plotted in Figure 6. Metadynamics is a powerful algorithm that can be used for both reconstructing the free energy and accelerating rare events in the systems described by complex Hamiltonians, at
the classical or at the quantum level (Laio and Gervasio 2008). In the algorithm the normal evolution of the system is biased by a history-dependent potential constructed as a sum of Gaussians centered along the distance between the E51 epoxy molecule and carbon fiber sheets in vertical direction. The sum of Gaussians is exploited for reconstructing iteratively an estimator of the free energy and forcing the system to escape from local minima (the adhesion state). Here the metadynamics method was used to calculate the free energy need for the system to change form adhesion state to detach state by summing the energy ejected into the system incessantly. Figure 6 (a-b) show the free energy profile of the interface model under different metadynamics simulation duration time. As show in Figure 6(a), 100ps simulation time are enough for the E51 epoxy to change from the energy minimum state--adhesion state (see in Figure 7(a)) to energy higher state-- detach state (see in Figure 7 (d)) in dry condition. With the simulation time increase, no more energy is added to the free energy profile for the change of states. When the metadynamics simulation time was 50ps or 80ps, the E51 epoxy was partly detached from the carbon fiber sheets (see in Figure 7(b-c)). In moisture condition, 50ps simulation time are enough for the E51 epoxy to change from the energy minimum state--adhesion state (see in Figure 7(e)) to energy higher state-- detach state(see in Figure 7 (h)). With the simulation time increase to 100ps, no more energy is added to the free energy profile for the change of states. When the metadynamics simulation time was 20ps or 30ps, the E51 epoxy was partly detached from the carbon fiber sheets (see in Figure 7(e-f)).

![Free energy profile](image1)

![Free energy profile](image2)

(c) The free energy profile of the interface in dry and water ingress condition

Figure 6 The free energy profile of the E51 epoxy/carbon fiber interface according to their vertical distance

The interfacial molecular adhesion can be estimated by calculating the free energy difference between the adsorbed state where the free energy was in the lowest value and the detached state where the curves have levels off. As shown, the binding free energy of CF/epoxy interface is around 43KJ/mol in dry condition and 17KJ/mol in water ingress condition.

In the adhesion state, the contact area between E51 epoxy molecules and carbon fiber sheets is around 3595Å² in dry condition and 2942 Å² in water ingress condition. Here we can calculate the surface energy by equation (1). \( S_0 \) is the contact area, \( E_b \) is the binding free energy. In dry condition, the surface energy is around 1.9mJ/mol. In moisture condition, the surface energy is around 0.96mJ/mol, with 49.7% reduction. Water molecules account for a significant part of the total work of adhesion of E51-epoxy/Carbon fiber interface by interaction with molecules of the interface. In the long-term application of FRP, small molecules of water would penetrate into the interface.
by diffusion and filtering through voids and cracks of the resin or by capillary migration along the fibers, thus the degradation of interface adhesion can’t be neglected.

\[ \gamma_s = \frac{E_b}{S_0} \]  

Figure 7 E51 epoxy molecules in different attach state from carbon fiber sheets during metadynamics simulation:  
(a) – (d) in dry condition; (e) – (h) in water ingress condition

As shown in Figure 8, carbon fiber is hydrophobic, with few water molecules permeated into its body. In our model, water molecules mainly interact with E51 epoxy molecules, increasing the chain movement of E51 epoxy molecules, making E51 epoxy molecules need less free energy to escape from the adhesive state with carbon fibers. In addition, water molecules would enter the area between carbon fibers and E51 epoxy molecules, forming hydrogen bonds with E51 epoxy molecules and decreasing the adhesion between E51 epoxy molecules and carbon fibers. Those two factors may account for the distance between the center mass of epoxy molecules and the surface of carbon fibers in vertical direction to change from 5.0 Å to 6.6 Å in stable adhesive states after changing from dry condition to wet condition. Thus lead to the decrease of Van der Waals’ force, which is in inverse proportion to distance. The binding free energy of E51 epoxy/carbon fibers is corresponding decrease in wet condition. Water molecules play an important role in the decrease of FRP interfacial bonding that can’t be neglected and is a main factor for the FRP debonding failure.
CONCLUSIONS

The interfacial mechanical behavior of E51 epoxy/Carbon fiber interface in dry and moisture conditions has been investigated using the microbonding pull-out test and molecular dynamics simulation. The simulation matched the experimental results well in showing the water ingress leads to remarkable degradation of interfacial bonding. The experiment shows that the interfacial shear strength of E51 epoxy/Carbon fiber interface could decrease to 59% after 90 days immersed in water. The adhesion of E51 epoxy carbon fiber decreased with the increase of duration of water immersion. The MD simulation shows the free energy for the E51 epoxy to debond from carbon fiber surface decreased to 51% of the free energy in the moisture condition. It is shown that water molecules account for a significant part of the total work of adhesion of E51-epoxy/Carbon fiber interface by interaction with molecules of the interface. In this way, the MD simulation gives us a qualified way to assess CF/epoxy interfacial adhesion by calculated the decrease of free energy and could enhance our understanding of failure mechanism in FRP.

As shown, the water ingress led to remarkable deterioration of CF/epoxy interfacial adhesion. There is a need to take the water effect into account during the design of FRP. Many applications of FRP are used in moisture environment and require FRP to work well for over 50 years. During the process, small molecules of water would penetrate into the interface by diffusion and filtering through voids and cracks of the resin or by capillary migration along the fibers, thus the degradation of interface adhesion can’t be neglected. The present study provides insights into the fiber-resin bonding mechanisms, and reveals the effect of water ingress on the bonding. The study is expected to be helpful for both the development of durable FRPs, and to predict the service life of a FRP subjected to hydrothermal aging accurately.

REFERENCES


FREEZE-THAW RESISTANCE OF WET LAY-UP BFRP LAMINATE AND BFRP-TO-BFRP ADHESIVELY-BONDED SINGLE-LAP JOINT

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ABSTRACT

Freeze-thaw (FT) cycling is one of the primary concerns for the application of fiber reinforced polymer (FRP) in cold regions. Existing studies have shown different levels of degradation on the mechanical behaviour of carbon FRP (CFRP) or glass FRP (GFRP) laminates and FRP-to-FRP adhesively-bonded joints after subjected to FT environment. This paper presents the FT durability test results of the basalt FRP (BFRP) and its adhesively-bonded single-lap joints. Standard FRP tensile coupons and FRP-to-FRP single-lap joints were prepared and tested before and after subjected to FT cycles. Micro-structure observations were also conducted to compare the fracture sections of the BFRP laminate with CFRP and GFRP laminates before and after FT exposure. Test results indicated excellent FT resistance for both BFRP laminate and its adhesively bonded single-lap joint. FT durability mechanism of BFRP composites and BFRP-to-BFRP adhesively bonded single-lap joint was discussed based on the experimental observations.

KEYWORDS

BFRP, freeze-thaw cycling, durability, single-lap joint.

INTRODUCTION

Fiber-reinforced polymer (FRP) has been successfully applied in a wide range of infrastructures because of their unique properties for strengthening or reinforcing of structures. The increasing use of FRP presents many challenges to researchers and engineers. One of these challenges is to understand the performance of FRP in harsh environments (Hollaway 2010). Freeze–thaw (FT) cycling is the primary types of environmental stress that seriously weaken the durability of concrete structures in cold regions (Sun et al. 2002). When subjected to FT cycling, thermal incompatibility of the constituent materials and the moisture environment may decrease the mechanical properties of FRP. Wet lay-up FRP laminates are common used in the field of strengthening of concrete structures. Existing studies have shown different levels of degradation on the tensile behaviour of carbon FRP (CFRP) or glass FRP (GFRP) laminates (Karbhari 2007, Li et al. 2012, Shi et al. 2014). Higher residual tensile strength in CFRP laminate has been observed than that in GFRP laminate after exposure to FT cycles.

The behaviour of adhesively-bonded joint is one of the critical issues in the FRP strengthened concrete structures especially for the FRP confined concrete columns. Service loads are transferred between fiber sheets within the area of FRP-to-FRP adhesive-bonded joint. Till now many studies have focused on this field and many analytical models have been developed for the prediction of the bond capacity of different adhesive-bonded joints under static loads and normal environments (e.g. Her 1999, Parvathy 2003, Li et al. 2009). However, the study on the FT durability of FRP-to-FRP adhesively-bonded joint is very sparse. Lopez-Anido et al. (2004) tested the FT resistance of prefabricated E-GFRP adhesive bonds and found substantial reduction on the lap shear strength after exposure to 20 FT cycles. However, little attention is given to the FT durability of wet lay-up FRP-to-FRP adhesive bond. As an emerging environmentally friendly material, basalt FRP (BFRP) has been shown to have superior advantages in structural retrofitting, seismic strengthening and serving as new structural components (Wu et al. 2012, Wang et al. 2013). Till now, very limited information is available on the durability of BFRP and BFRP-to-BFRP adhesively-bonded joint in FT environments.

This paper introduces the durability study of BFRP laminate and BFRP-to-BFRP adhesively-bonded single-lap joints subjected to FT cycling. Standard FRP tensile coupons and FRP-to-FRP single-lap joints were prepared and
tested before and after subjected to FT cycles. Micro-structure observations were also conducted to further understand the FT resistance of BFRP and compare that with GFRP and CFRP. FT durability mechanism of BFRP and BFRP-to-BFRP adhesively bonded single-lap joint was also discussed based on the test results.

EXPERIMENTAL PROGRAM

Test coupons

To make BFRP tensile coupons, one layer of unidirectional basalt fiber sheet was completely impregnated with epoxy resin using a wet hand lay-up procedure. The nominal thickness of the basalt fiber sheet is 0.156mm. After the solidification of the epoxy resin, BFRP coupons were cut and prepared according to the GB/T-3354 (1999), as shown in Figure 1 (a). The FT durability test of corresponding epoxy resin was also conducted to reflect the degradation mechanism. Epoxy resin tensile and tensile shear test coupons were prepared according to the GB/T-2567 (2008), as shown in Figure 1 (b) and (c). BFRP-to-BFRP adhesively-bonded single-lap shear coupons were prepared based on the reference to the previous studies (Parvathy 2003, Li et al. 2009) with one layer of basalt fiber sheet bonded to the same other one, as show in Figure 1 (d). The overlap length (OL) used in the tests is 100 and 120mm for the two groups of BFRP-to-BFRP test coupons. All of the test coupons were cured for a minimum of one week at ambient temperature prior to testing.

![Figure 1 Test coupons in this study: (a) BFRP laminate; (b) Epoxy resin (tensile coupon); and (c) Epoxy resin (shear coupon); (d) BFRP-to-BFRP single-lap joint (Units in mm)](image)

Test procedures

The test coupons were soaked in water in rubber boxes and subjected to up to 200 accelerated FT cycles. The freezing and thawing condition were conducted at 3-4 hours per cycle, in accordance with the rapid FT method in the Chinese national standard GB/T 50082-2009. The temperature fluctuation is ranged from +8°C to -17°C during FT tests. After being subjected to scheduled FT cycles, each coupon was positioned in a 30kN tension testing machine for the tensile tests. The axial deformation of BFRP and epoxy resin tensile coupons was recorded using an extensometer with a gauge length of 50 mm. The tensile force was measured by a load sensor inside the testing equipment. The loading was controlled by displacement at a rate of 1.0 mm/min. The values of the load and the axial deformation were recorded at one-second intervals.

TEST RESULTS AND DISCUSSIONS

Test results of BFRP laminate and epoxy resin

Table 1 summarizes the mechanical properties of the BFRP laminate and epoxy resin at room temperature. Each value in the table is the average of 6 identical coupons, and the standard deviation is also given after the symbol “±”. The tensile properties of the BFRP laminate are calculated based on the nominal thickness (i.e., without considering the thickness of the resin matrix).

<table>
<thead>
<tr>
<th>Test coupons</th>
<th>Tensile strength (MPa)</th>
<th>Elastic modulus (GPa)</th>
<th>Rupture elongation (%)</th>
<th>Shear strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BFRP</td>
<td>2077 ± 86</td>
<td>80.2 ± 1.5</td>
<td>2.68 ± 0.01</td>
<td>-</td>
</tr>
<tr>
<td>Epoxy resin</td>
<td>47.1 ± 5.1</td>
<td>3.2 ± 0.3</td>
<td>2.39 ± 0.44</td>
<td>15.1 ± 1.3</td>
</tr>
</tbody>
</table>

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Figure 2 illustrates the variation in the residual mechanical properties of the BFRP laminate and the corresponding epoxy resin with respect to the exposure FT cycles. To simplify the analysis and facilitate comparison of the results after FT exposure, the test results were expressed by normalized values with respect to the control test results. The error bars in the figure represent the coefficient of variation (CV) of the test data. The tensile strength of the BFRP laminate decreased slightly at first 50 cycles, then increased up to 150 cycles, and finally decreased slightly. There was almost no reduction in the residual tensile strength of the BFRP laminate after 200 FT cycles. In contrast, nearly 20% degradation was observed on the tensile strength of epoxy resin after 100 and 200 FT cycles. The FT cycling did not diminish the elastic modulus of the BFRP laminate and the epoxy resin. A maximum increase in the elastic modulus of approximately 10% was observed for BFRP, indicating that FT cycling can contribute brittleness to the BFRP laminate. The rupture elongation of the BFRP laminate and the epoxy resin decreased as the number of FT cycles increased. After 200 cycles, the rupture elongation of BFRP laminate is around 90% of the control ones, which is mainly due to the increased brittleness introduced by FT cycling. Figure 2 (d) shows that the shear strength of the epoxy resin decreased steadily as the number of FT cycles increased. Up to 100 cycles, there was no significant difference in the degradation rate in comparison to that of the tensile strength of the epoxy resin. At a higher number of cycles, the normalized values of the tensile shear strength were much lower than the corresponding tensile strength values. The above analysis indicates that although the epoxy resin matrix was damaged, the tensile properties of BFRP laminate were almost not influenced by the FT cycling.

Figure 3 compares the micro-structure on the fracture sections of the BFRP laminate before and after FT exposure by scanning electron microscope (SEM) observations. Masses of matrix fragments can be found on the surface of basalt fibers after tensile fracture, as shown in Figure 3 (a), indicating good bonding condition between resin matrix and fibers for the control BFRP coupons. In comparison, relative lesser matrix fragment was observed on basalt fiber surface, as shown in Figure 3 (b). This is mainly due to the bond degradation of epoxy resin after FT cycling. To further study the effect of fiber types and epoxy resin degradation on the durability of FRP composites, the 200 FT cycles exposed CFRP and GFRP laminates were also conducted through the SEM observation testing. Obvious debonding phenomenon can be found between carbon or glass fibers and resin matrix, as shown in Figure 3 (c) and (d), indicating that the fiber-matrix bond performance was damaged by FT cycling in some levels. In comparison, the fraction section of BFRP laminate did not shown this kind of clear debonding between fiber and matrix.
Test results of BFRP-to-BFRP single-lap joints

Figure 4 shows the typical failure mode of the BFRP-to-BFRP single-lap joint. Debonding delamination between the two layers of basalt fiber sheets within the overlap section of the test coupons can be observed for all of the tested single-lap coupons in this study.

Figure 5 shows the normalized results of the apparent shear strength of BFRP-to-BFRP single-lap joints after subjected to different FT cycles. The apparent shear strength ($S$) of the single-lap joint is determined by dividing the peak load, $P$, by the overlap area, $A_L$. In which, $A_L$ is calculated by multiplying the measured coupon width, $b$, by the overlap length, $O_L$. Two groups of the test coupons, B-100 and B-120, with $O_L=100\text{mm}$ and $120\text{mm}$ were carried out to study the effect of overlap length on the FT durability of BFRP-to-BFRP adhesive bond capacity. The average value of $S$ is $2.92\text{MPa}$ and $2.41\text{MPa}$ for the test coupons B-100 and B-120, respectively. As can be observed from Figure 5, the apparent shear strength almost did not degrade with the increased FT cycles for both
B-100 and B-120 test coupons. After 200 FT cycles, the normalized average value of $S$ is 1.01 and 1.11 for B-100 and B-120, respectively. B-120 coupons retained higher shear strength than B-100 coupons, indicating that the longer overlap length can result in better FT resistance for the bond performance of BFRP-to-BFRP adhesive-bonded joints. Overall, BFRP-to-BFRP adhesive-bonded joints behave satisfactory durability in FT cycling environment.

**Discussions of the FT durability mechanism**

**BFRP laminates**

Two major effects occur when FRP composites are exposed to FT environments: thermal incompatibility and degradation of the constituent materials, including their interfaces (Green 2007). The first effect is related to the different thermal characteristics of the constituents of FRP composites. A decrease in temperature would cause the fibers and resin to contract, resulting in the formation of residual stresses at the resin matrix-fiber interface, due to the differences in the coefficients of thermal expansion (CTE) and the elastic modulus between the two materials. Basalt fibers have closer CTE to the epoxy resin than the carbon or glass fibers. Hence, better bonding performance between the basalt fibers and matrix after FT cycling can be achieved. This can be confirmed by the SEM analysis as discussed previously (Figure 3). The other major effect is the degradation of the constituent materials and their interfaces as a result of FT exposure, especially in the presence of moisture. Exposure to cold temperatures and FT cycles generally results in an increase in the elastic modulus and embrittlement of a resin matrix but a decrease in damage tolerance, contributing to the increase in the elastic modulus and the decrease in the tensile strength and rupture elongation of the FRP composites. According to previous studies (Karbhari 2007, Shi et al. 2014), the tensile strength of FRP composites is sensitive to the bond between fiber and resin matrix. As a result, good bond performance of the basalt fiber-matrix interface can lead to satisfactory durability of BFRP laminate in FT cycling environment.

**BFRP-to-BFRP adhesive-bonded joints**

The FT durability of prefabricated FRP-to-FRP adhesive bonds is mainly determined by the FT resistance of bonding adhesives. As the shear strength of epoxy resin decreases significantly with FT cycles, substantial reduction on the lap shear strength can be found on the prefabricated FRP-to-FRP adhesive-bonded joints (Lopez-Anido et al. 2004). For the condition of wet lay-up FRP-to-FRP adhesive-bonded joints, the adhesive layer between the two fiber sheets is very thin. The debonding of fiber and matrix within the area of overlap length controls the joint failure. As a result, good bond performance for the basalt fiber-matrix interface can also lead to a good resistance to the FT cycling for BFRP-to-BFRP adhesive-bonded joints.

**CONCLUSIONS**

Based on the results of the experimental studies and the discussions presented in this study, the following conclusions can be drawn:

1. FT cycling has a negligible effect on the tensile properties of BFRP laminate, while significant degradation of the mechanical properties of epoxy resin was observed after FT exposure.
2. FT cycling did not degrade the apparent shear strength of BFRP-to-BFRP adhesively-bonded single-lap joints. Longer overlap length can result in better BFRP-to-BFRP bond FT resistance.
3. Good bond performance of the basalt fiber-matrix interface is the main reason for the satisfactory FT resistance of both BFRP laminate and BFRP-to-BFRP adhesively-bonded single-lap joint presented in this study.

**ACKNOWLEDGMENTS**

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**REFERENCES**


INFLUENCE OF CONCRETE CHEMISTRY ON CFRP BONDING EPOXY LONG TERM PROPERTIES

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²EDF Lab MMC, Les Renardières - Ecuelles, France

ABSTRACT

The performance of CFRP used in RC structures reinforcement resides in the ability of the adhesive bond to effectively transfer the load from the concrete substrate to the CFRP and also to maintain this behaviour through the structure service life.

The current study focused on the influence of the concrete alkaline chemical nature on the epoxy network properties. Thus, the ageing of epoxy resins from 2 different types of CFRP reinforcement has been studied after thermo-oxidative exposure and alkaline solution immersion. Both chemical and mechanical properties evolutions have been characterized.

The epoxy network oxidation kinetic was compared between thermo-oxidative atmosphere and alkaline concrete environment. The influence of water intake, alkaline degradation reactions or hydrolysis on the epoxy network mechanical properties was investigated.

Based on these experimental results and the identified chemical ageing mechanisms, a Polymer Ageing Prediction numerical tool was evaluated.

KEYWORDS

FRP, epoxy ageing, thermo oxidative, alkaline, adhesive bond, concrete.

INTRODUCTION

Carbon Fiber Reinforced Polymers have been commonly used worldwide for the strengthening and repair of civil structures for decades now. If the technique effectiveness has been clearly demonstrated and is now accepted by all decision makers in the construction sector, a major concern is the durability of the repairs. Indeed, the long-term mechanical behaviour of concrete/composite adhesively bonded interfaces exposed to mechanical and environmental ageing mechanisms throughout the life cycle of the structure has not yet been fully demonstrated. Over the last decade, several authors have reported that environmental conditions may indeed affect the integrity of both the interfacial bonds (substrate/polymer interactions) and the polymer material itself (adhesive joint in the case of bonded composite plates).

Accelerated conditions such as freeze-thaw/dry-heat cycles, immersion in salt solutions or exposure to humid environments were found to be detrimental to the adhesive bond properties (Karbhari et al 2007, Grace & Singh 2005, Benzarti et al. 2011). Also, the few available literature on field service study of CFRP strengthening indicated significative evolution of interfacial failure mode and defects propagation (Allen 2011, Benzarti et al 2009). The current study focused on the influence of the concrete alkaline chemical nature on the epoxy adhesive bond properties. Thus, the ageing of epoxy resins from 2 different types of CFRP reinforcement has been studied after thermo-oxidative exposure and alkaline solution immersion.

EXPERIMENTAL APPROACH

Materials

Two commercial epoxy resin were studied, and will be referred to as EPX A and EPX B. EPX A. The EPX A is commercially sold as an adhesive used as a wet-lay up applied solution of Carbon Fiber/Epoxy resin composite on concrete. The EPX B is used as an adhesive for pultruded CFRP plates glued on concrete. Table 1 shows that the
two selected epoxy resins are fairly different in composition and stiffness. The micrographs presented in the Figure 1 confirms these differences, displaying the EPX B high coarse fillers content.

Table 1 Materials

<table>
<thead>
<tr>
<th>Reference</th>
<th>Type</th>
<th>Fillers</th>
<th>Modulus (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EPX A</td>
<td>Epoxy adhesive</td>
<td>&lt;15% fine silica</td>
<td>3.4</td>
</tr>
<tr>
<td>EPX B</td>
<td>Epoxy adhesive</td>
<td>&gt;80% coarse silica</td>
<td>12.5</td>
</tr>
</tbody>
</table>

Both resins were processed manually with a 150x150x4 mm³ PTFE mold before being aged and sampled for characterization. The minimum curing time at 20°C and 50% HR was 1 week.

![Micrographs of polished plate’s cross section of both epoxy resin EPX A and EPX B after processing](image)

Figure 1 Micrographs of polished plate’s cross section of both epoxy resin EPX A and EPX B after processing

**Accelerated ageing conditions**

The goal of this study was to compare the ageing mechanism between a classic thermo-oxydative environment and the alkaline reactive medium in contact with concrete. Thus, two accelerated ageing set-up were utilized:

- Thermo-oxydative conditions: aging in ventilated stoves at 70°C, 90°C and 110°C for up to 6000h,
- Cementitious conditions: immersion in a saturated lime solution at 40°C, 60°C and 80°C for up to 6000h.

**Characterization**

The vitrous transition temperature was determined using a NETZSCH DMA 242 device. Infra-red analysis was performed with a BRUKER FTIR microscope allowing one measurement every 128 µm. Tensile testing was performed according to ISO 527 with a INSTRON 8872 dynamometer on ISO 527 H2 dumbbells samples.

**RESULTS AND DISCUSSIONS**

**Influence of thermo-oxydative ageing**

The vitrous transition temperature Tg of the EPX A reference decreases at first until 500-1000h of accelerated ageing before increasing for longer times as shown in Figure 2. Concerning the EPX B, the Tg quickly increases and stabilizes. Both resins’ Tg decreases for the longest ageing durations. The lowering of Tg can be explained by crosslink openings whereas the increase is probably due to both post-curing and additional cross-kinking even though additional characterisation is needed in so far as only one sample per ageing step was tested. Previous studies also showed a competition between crosslink bond cleavage and additional crosslinking in epoxy thermo-oxydative ageing mechanisms (Zahra et al. 2014).
The chemical functions evolution through ageing was studied with FT-IR microscopy. The thermo-oxidative ageing mechanism of epoxy resin is already well documented and it has been shown that the carbonyl species concentrations are representative of the oxidation progression. Figure 3 shows both EPX A and EPX B 1710 cm⁻¹ carbonyl peak absorbance as a function of depth and ageing duration. All spectra were normalized with the 1608 cm⁻¹ peak absorbance attributed to aromatic cycles which are known not to be impacted by oxidation mechanisms (Zahra et al. 2014).

Figure 2 Evolution of T<sub>g</sub> as a function of thermo-oxidative ageing duration

The carbonyl peak absorbance being proportional to carbonyl species concentration and thus to resin degradation through oxidative ageing, the Figure 3 a) shows a oxygen diffusion limited mechanism as the area closest to the sample surface are the most rapidly oxidized. Also the progressive increase through ageing time is clearly visible. Not presented in this paper, the influence of ageing temperature on the carbonyl production kinetic was observed. The oxidized layer thickness is estimated at about 1 mm. Unfortunately, the EPX B high fillers proportion impeaches a good spectra resolution, and is not fully exploitable. Still, the increase of carbonyl concentration is also visible for the EPX B resin.

Tensile mechanical properties characterization results for EPX A showed a tensile modulus decrease for the lowest ageing temperature of 70°C until 2000h. From 2000h to 4000h the tensile modulus started to increase before dropping between 4000h and 6000h. For higher ageing temperatures, the tensile modulus increased until 4000h before dropping for further ageing duration. Elongation at break did not display any clear tendencies other than a
general increase for the lowest ageing temperature and a decrease for the highest ageing temperature. The tensile modulus increase can be explained by additional crosslink bond formation. Incidentally a tensile modulus decrease could be linked to crosslink or backbone bonds cleavage. As both modulus increase and decrease occurs alternatively depending on the ageing temperature or duration, a competition between both bond formation and cleavage is likely explain the mechanical behavior of the EPX A epoxy resin through thermo-oxydative ageing. This hypothesis is backed with the Tg evolution results depicted before.

Concerning the EPX B resin, the tensile modulus showed a small decrease through ageing time for all temperatures. The elongation at break decreased rapidly before stabilizing at about half the pristine state value. The lower amplitude of the influence of thermo-oxidative ageing on the EPX B resin might be caused by the high concentration of fillers (>80%) in said resin. Incidentally, the elongation at break decrease is probably linked to filler/matrix chemical bond degradation through thermal ageing.

![Figure 4 Mechanical properties after tensile testing after thermo-oxidative ageing](image)

**Influence of cementitious environment ageing**

The diffusion coefficient of cementitious water through both EPX and EPX B epoxy network were determined (Table 2) using a Fickian model (Equation 1).

\[
D(T) = A e^{-\frac{E_a}{RT}}
\]  

(1)
Table 2 Cementitious water diffusion parameters

<table>
<thead>
<tr>
<th>Reference</th>
<th>$A$ (m²/s)</th>
<th>$E_a$ (kJ/mol)</th>
<th>$R$ (J/mol)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EPX A</td>
<td>$2.95 \times 10^7$</td>
<td>70.6</td>
<td></td>
</tr>
<tr>
<td>EPX B</td>
<td>$5.9 \times 10^4$</td>
<td>63, 1</td>
<td>8.314</td>
</tr>
</tbody>
</table>

The results show that diffusion kinetics of cementitious water are much faster in EPX A resin than EPX B resin. This can be explained by the high fillers content of EPX B compared to EPX A.

Cementitious environment aging led to a fast drop of $T_g$ for EPX A and EPX B at 80°C. For longer ageing times, both EPX A and EPX B samples displayed a decrease in $T_g$.

Figure 5 Evolution of $T_g$ as a function of cementitious environment ageing duration

Figure 6 FT-IR microscopy showing the evolution of 1710 cm⁻¹ carbonyl and 3350 cm⁻¹ hydroxyl characteristic peaks as a function of sample depth and cementitious water ageing duration at 80°C
In addition to carbonyl species, hydroxyl groups hypothetically produced by hydrolysis reaction (Zhang et al. 2012) were monitored with FT-IR microscopy. The infra-red peak associated to hydroxyl groups is located around 3350 cm\(^{-1}\). The results presented by the Figure 6 show hydroxyl group concentration increase through ageing time for the EPX A sample, meaning that hydrolysis mechanisms do take place. Also, as opposed to thermo-oxidative mechanism, cementitious water induced chemical modifications seems homogeneous in the sample thickness. The carbonyl groups, however, tend to disappear with ageing duration, although the typical color change is witnessed on the aged samples. A potential explanation is a lixiviation phenomenon of carbonyl species by the cementitious water. The EPX B sample was not characterized because of the filler induced poor signal quality, as seen previously.

Tensile mechanical properties characterization results for both EPX A and EPX B after cementitious water aging showed a steep tensile modulus drop in the first 200h of aging (Figure 7). The elongation at break for EPX A also rose quickly. This tendency can be explained by the water intake, acting as a plasticizer in the resin epoxy network. EPX A tensile modulus later increased until 3000h of ageing, whereas the elongation at break decreased slightly. This can be understood as additional crosslinking of the epoxy network, in a similar way as in thermo-oxidative environment. EPX B tensile modulus stays stable after the initial drop, whereas the elongation at break continues to rise. This can be explained by the high filler content of this resin, leading to matrix/fillers interface debonding phenomenon unrelated to the matrix alone chemical modifications.

Ageing mechanism model

In order to simulate the ageing mechanisms of polymer networks, a numerical tool has been elaborated (Zahra 2012) and will not be detailed in this paper. Although the model has been validated in 1D virtual polymer networks, the O\(_2\) diffusion is still not fully implemented. However, first simulations show promising results as the oxidation...
layer is well reproduced. The Figure 8 shows the carbonyle concentration [CO] as function of sample depth and thermo-oxidative ageing duration à 110°C. The future work includes parameterization of the model according to the experimental results proposed in this work in terms of ageing kinetics, water intake as a function of temperature and time.

![Figure 8 Numerical simulation of carbonyle concentration through 110°C thermo-oxidative ageing in a model polymer](image)

**CONCLUSIONS**

This experimental study showed that both epoxy resins underwent oxygen diffusion limited oxidation in thermo-oxidative atmosphere leading to a maximal tensile properties decrease. The cementitious water ageing induced oxidation and hydrolysis mechanisms, although in lower proportion than the thermo-oxidative conditions showed. Hence, the mechanical properties evolutions are predominately caused by the water intake, acting as a plasticizer in the epoxy network. Cementitious water intake diffusion kinetics was determined. Tensile modulus values dropped up to 5 times after only 200h immersion, while elongation at break drastically increased. The studied resins being used as civil structures strengthening, both thermo-oxidative and cementitious environment ageing are to be taken into account, as this work’s conclusions showed the great influence of both ageing environment on the resins mechanical properties. The plasticizer effect of both external source and cement water cannot be overseen when considering the composite creep behavior, as shown in other studies (Houhou 2014). The homemade numerical model, enriched with this study results, will be of great help in lifetime prediction of such strengthening systems.

**REFERENCES**


Zhang Y (2012). A spectroscopic study of the degradation of polyurethane coil coatings, Ph D, School of Engineering and Material Science, Beijing, China.
ABSTRACT

The use of externally-bonded fibre-reinforced polymer (FRP) composites to strengthen and repair existing concrete structures is an established technology. In this technology, organic adhesives (epoxy and other resins) are widely applied, since they have been proven to be excellent bonding agents between the FRP plates or sheets and the substrate materials (i.e. concrete, metal, wood) under common operation conditions and room temperatures. However, the poor temperature-resistance performance of epoxy is an obstacle for the application of FRP in civil buildings, such as residences, hospitals and commercial centres, which usually have a certain period of time of fire resistance requirements. The cementitious bonding agent is supposed to be a practical alternative of organic adhesives to solve this shortage. This paper therefore presents the performance at elevated temperatures of cement-based carbon fibre-reinforced composite to strengthen concrete structures. More specifically, a series of single-shear FRP-to-concrete joints using mortar as bonding agent were tested at steady state for temperatures ranging from approximately 20 to 700℃. The failure modes and the ultimate capacities at elevated temperatures were recorded. The bond strength of FRP-to-concrete joints at such high temperatures was compared with that at room temperature.

KEYWORDS

FRP, elevated temperatures, concrete structures, mortar, high-temperature behaviour.

INTRODUCTION

In current CFRP strengthening systems, the bond agents used as the matrix and also used as bond adhesive between the FRP composites and the substrate materials (i.e. concrete, metal and wood), are usually epoxies. Present-day epoxy resin products all basically consist of two components that react with each other forming a hard, inert material. One part consists of an epoxy resin and the other part is the epoxy curing agent, sometimes called hardener. The epoxies, as organic matter, degrade with increased temperature. Such degradation commences before the glass transition temperature, \( T_g \), is reached and subsequently the reductions of the bond properties occur. Normally, the \( T_g \) can lie in the range of 50 to 90℃ for commercially available products used in civil infrastructure applications (Foster et al. 2008). However, such temperatures can be easily exceeded when there are fires in buildings of residences, hospitals and commercial centres. Thus, the poor temperature performance of epoxy has been an obstacle for the applications of FRP in civil buildings, which usually have a certain period of time of fire resistance requirements.

Most recently, a cost effective way has been attempted to solve the fire-resistance shortage of the current FRP technology, in which a cement-based bonding agent is proposed as a practical alternative to organic adhesives. The called cement-based bonding agent is the common constructional material, mortar, which is one component, adds polymer reinforced powder material mixed with water. In addition to higher fire resistance and lower cost, it also has several advantages over the epoxy resins including no harmful toxicity to human body, better thermal compatibility with the concrete substrate and good operability at low temperatures (Hashemi et al.; 2008; Blanksvård et al. 2009). Of the limited research undertaken to date on cement-based FRP strengthened concrete structures, full scale beams and columns tests (Hashemi et al.; 2008; Blanksvård et al. 2009; Wiberg 2003; Täljsten et al. 2007; Wu et al. 2010; Badanoiu et al. 2003) have been conducted. These test results have shown that the sufficient mechanical bond properties for load transfer could be obtained using mortar. Such testing, however, has only considered the behaviour of the entire strengthened member or joints strengthened by cement-based FRP systems at room temperatures and not at elevated temperatures.
This paper reports the results of a series of single shear tests on the fundamental behaviour of FRP-to-concrete joints using mortar as a bonding agent at temperatures ranging from room temperature (approximately 22°C) up to high temperatures which may be expected in a fire (i.e. 700°C). It is important to note here, there are many types of commercial modified cements available, which will affect the test results more or less. Such study, however, is outside the scope of this paper. The cement-based bonding agents in this paper were prepared by just using most common commercially cement-based materials, superfine cement (SFC). By using a set of universal testing machines, the test specimens were heated to a specified temperature and then the tensile test carried out. The behaviour of the test specimens are reported as well as the failure modes.

EXPERIMENTAL INVESTIGATION

Test Set-up, Instrumentation and Loading

Figure 1 shows the overall view of test set-up. To restrain the displacement of the concrete block, a steel holding frame for the single-shear pull tests was fabricated and mounted inside a MTS 810 universal testing machine. Prior to fastening the bolts of this steel frame, the concrete block was positioned by adjusting the front baffle plate, which placed the FRP plate in a uniform line with the tensile force applied by the testing machine. The heating device used was an MTS Model 653 high temperature furnace with a maximum temperature of 1400°C. One side of the furnace which has internal thermocouples inside was used in this study. The temperatures were controlled by an MTS 409.83 temperature controller. The positions of the furnace and the FRP plate are shown in Figure 2 and maintained consistently for all 24 joints. The external thermocouples were put in the furnace and contacted the surface temperature of FRP plates, the readings of which were considered as the real temperatures of the specimens in this paper.

In the testing procedure, after a specimen was fixed, the temperature was raised to the desired temperature and then held constant for a target period of 30 or 90 minutes to allow the mortar or epoxy to undergo physical and chemical changes. During these stages, the free end of the FRP plate was unrestrained in order to enable the thermal expansion. After the holding time had been reached, the top jaws of the MTS were closed and then the tensile loads were applied on the specimens (while still under the influence of temperature) to failure.
Materials

Superfine cement (SFC) was used to make the mortar in this study. SFC has a mean particle size of 3.1\(\mu\)m as compared to 12.4\(\mu\)m for ordinary Portland cement (OPC). Table 1 presents the mixing ratios of the evaluated mortars in the laboratory test. In the mixing ratio, as superplasticizer (SP) was added to each mortar in order to disperse the cement grains and reduce agglomeration. The SP used was a polycarboxylate ether-based polymer having a molecular structure characterized by a main chain attached with side chains (ASTM C 109/C 109M, 2007). The silica fume (imported from Norway) was used as another addition to change the microstructure of the mortar and decrease the porosity of the transition zone. Table 2 presents overview of the mechanical properties of the evaluated mortars in the tests. The test method for the compressive strength of the mortars used 50 mm cube specimens according to ASTM C109/C109M-07, meanwhile, the elastic modulus was found according to ASTM C 469-94. The flexural strength was found according to ASTM C 348-02.

The commercial concrete was adopted and poured in one batch for all of the concrete prisms. All these samples were kept in a controlled environment over ten months, then prepared for bonding the FRP plate and tested in two and a half months. Concrete material property tests were conducted before starting the single-shear tests. The resulting properties were cube compressive strengths of 46.3 MPa; the other material properties of concrete block are also listed in Table 2.

The joints were divided into 4 groups (G1-G4), namely (i) G1 Room temperature Groups, (ii) G2 Time Groups, (iii) G3SFC Groups, and (iv) G4 Epoxy Groups. The materials, holding times and testing temperatures of each group are summarized in Table 3. The first group is the room temperature which is no holding temperature process in testing. The next two groups of joints G2 and G3 used the mortar as bond agent, whereas G4 used epoxy as bond agent. The holding time of the joints in G2 and G3 had the target holding time of 30 and 90 minutes, respectively.

### Table 1. Mix ratio of mortars

<table>
<thead>
<tr>
<th></th>
<th>Cement (kg)</th>
<th>Water (kg)</th>
<th>Silica fume (kg)</th>
<th>SP (kg)</th>
<th>Water/Binder (^1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SFC</td>
<td>1200</td>
<td>396</td>
<td>120</td>
<td>29.2</td>
<td>0.4</td>
</tr>
</tbody>
</table>

\(^1\)Binder = Cement + silica fume

### Table 2. Material and mechanical properties for evaluated mortars and concrete

<table>
<thead>
<tr>
<th></th>
<th>Density (g/cm(^3))</th>
<th>Compressive strength (MPa)</th>
<th>Splitting Strength (MPa)</th>
<th>Flexural strength (MPa)</th>
<th>Modulus of Elasticity (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SFC Mortar</td>
<td>1.77</td>
<td>16</td>
<td>-</td>
<td>1.1</td>
<td>12.3</td>
</tr>
<tr>
<td>Concrete Block</td>
<td>2.52</td>
<td>46.3</td>
<td>3.41</td>
<td>4.17</td>
<td>29.7</td>
</tr>
</tbody>
</table>

Experimental results

A detailed description of the performance and failure modes of all test specimens is presented in this section. The behaviour of bond agent using mortar at room temperature and elevated temperatures was compared with the epoxy. A summary of the strength, testing temperature and failure modes of all specimen results contained in Table 3.

### Room temperature

The average ultimate loads of three types of FRP joints (using SFC and epoxy as bond agents) are 9.48 and 15.34 kN, respectively. The ultimate capacities of the joints using SFC as bond agents were around two-thirds of the joints using epoxy as the bonding agent. The joints of the epoxy failed in an explosive way by debonding of the concrete at the interface with the adhesive, leaving 1-2 mm of concrete attached to the CFRP plate and adhesive. This type of failure mode was shown widely by previous tests. Main failure modes of SFC were delamination between the fibers of plate. The reason for this is mainly because the mortar of SFC can not fully penetrate the fabric effectively. The quality of the bond capacity using mortar as the bonding agent is affected more vulnerably.
by manual operations than epoxy. It should be noted that the aim of this study is not to investigate the behaviour of the FRP at room temperature. Therefore, subsequent studies at room temperature which address extensive parameters such as bond length, FRP width, epoxy etc., are qualitative.

The effect of holding time

The results of Group G2 tests show that the ultimate strengths and failure modes for the specimens were almost the same as compared to the corresponding specimens in Group G3. Such results are shown in Tables 3. The reason for this behaviour was that the cementitious material was static under such elevated temperatures and holding time did not affect the ultimate load of these single shear joints.

Behaviour and failure modes at elevated temperatures

A brief description of all failure modes encountered in all tests is provided in Table 3 with selected typical photos of failed joints presented in Figure 3. The reduction factor of the ultimate strength at temperature $T$ to the ultimate strength at room temperature ($P_{u,T}/P_{u,normal}$) obtained from the steady state tests at different temperatures is plotted in Figure 4.

When the temperature rose to around $150\,^\circ C$, the test joints of epoxy series experienced a significant drop in strength compared with the room temperature strength. From the Figure 4, though the mortar has a weaker bond properties compared with the epoxy at room temperature, this superiority will not exist for the epoxy after around $150\,^\circ C$. At $150\,^\circ C$, the ultimate load of the mortar series was obvious higher than the ultimate load of the epoxy series.

For epoxy series, the reduction tendency of the ultimate strength was very obvious at selected levels of temperature. Referring to the joints tested at room temperature, the reduction of the ultimate strength was 0.72 at $130\,^\circ C$, 0.54 at $150\,^\circ C$ and 0.44 (the mean value) at $200\,^\circ C$. Three different types of failure modes were found in the tests (see Table 3). At $22\,^\circ C$, failure occurred in the concrete, leaving about 1-2 mm of concrete attached to the unbounded CFRP plates. At higher temperatures ($130\,^\circ C$ and $150\,^\circ C$) debonding occurred in the adhesive, leaving about 1 mm of epoxy attached to the unbounded CFRP plates. At $200\,^\circ C$, debonding occurred in the adhesive and FRP plate.
interface, leaving hardly any epoxy attached to the CFRP plate. For both failure modes debonding occurred in a rather explosive way.

From the Figure 4, it can be seen that before 600°C, the ultimate loads of the joints using mortar as the bonding agent showed no obvious reduction at elevated temperatures. The ultimate load of joint M-700-90-1 and M-600-90-1 was 0.39 and 0.69 of ultimate loads at room temperature, respectively.

### Table 3. Selected test results

<table>
<thead>
<tr>
<th>Group</th>
<th>Specimen Identification¹</th>
<th>Bond agent</th>
<th>Temperature of FRP Surface, T (°C)</th>
<th>Holding Time (min)</th>
<th>Ultimate Load, fu,t(kN)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>G1</td>
<td>M-23-0-1 Mortar</td>
<td>23</td>
<td>-</td>
<td>8.80</td>
<td>DB-MFI</td>
<td></td>
</tr>
<tr>
<td></td>
<td>M-23-0-2 Mortar</td>
<td>23</td>
<td>-</td>
<td>10.41</td>
<td>DB-CMI and DB-MFI</td>
<td></td>
</tr>
<tr>
<td></td>
<td>M-23-0-3 Mortar</td>
<td>23</td>
<td>-</td>
<td>9.25</td>
<td>DF</td>
<td></td>
</tr>
<tr>
<td></td>
<td>E-23-0-1 Epoxy</td>
<td>23</td>
<td>-</td>
<td>16.67</td>
<td>DB-CAI</td>
<td></td>
</tr>
<tr>
<td></td>
<td>E-23-0-2 Epoxy</td>
<td>23</td>
<td>-</td>
<td>14.21</td>
<td>DB-CAI</td>
<td></td>
</tr>
<tr>
<td>G2</td>
<td>M-100-30-1 Mortar</td>
<td>100</td>
<td>27</td>
<td>9.88</td>
<td>DB-CMI</td>
<td></td>
</tr>
<tr>
<td></td>
<td>M-200-30-2 Mortar</td>
<td>205</td>
<td>25</td>
<td>10.32</td>
<td>DB-CMI</td>
<td></td>
</tr>
<tr>
<td></td>
<td>M-300-30-1 Mortar</td>
<td>302</td>
<td>25</td>
<td>11.02</td>
<td>DB-CMI</td>
<td></td>
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<tr>
<td></td>
<td>M-300-30-2 Mortar</td>
<td>304</td>
<td>32</td>
<td>8.89</td>
<td>DB-CMI</td>
<td></td>
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<tr>
<td></td>
<td>M-400-30-1 Mortar</td>
<td>403</td>
<td>24</td>
<td>11.35</td>
<td>DB-CMI</td>
<td></td>
</tr>
<tr>
<td></td>
<td>M-400-30-2 Mortar</td>
<td>405</td>
<td>25</td>
<td>9.69</td>
<td>DB-CMI</td>
<td></td>
</tr>
<tr>
<td></td>
<td>M-500-30-1 Mortar</td>
<td>525</td>
<td>32</td>
<td>11.41</td>
<td>DB-CMI and DF</td>
<td></td>
</tr>
<tr>
<td></td>
<td>M-600-30-1 Mortar</td>
<td>600</td>
<td>30</td>
<td>8.93</td>
<td>DB-CMI</td>
<td></td>
</tr>
<tr>
<td>G3</td>
<td>M-100-90-1 Mortar</td>
<td>100</td>
<td>90</td>
<td>10.54</td>
<td>DB-CMI</td>
<td></td>
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<tr>
<td></td>
<td>M-200-90-1 Mortar</td>
<td>200</td>
<td>90</td>
<td>10.48</td>
<td>DB-CMI</td>
<td></td>
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<tr>
<td></td>
<td>M-300-90-1 Mortar</td>
<td>300</td>
<td>90</td>
<td>10.68</td>
<td>DB-CMI</td>
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</tr>
<tr>
<td></td>
<td>M-400-90-1 Mortar</td>
<td>400</td>
<td>90</td>
<td>10.01</td>
<td>DB-CMI</td>
<td></td>
</tr>
<tr>
<td></td>
<td>M-500-90-1 Mortar</td>
<td>530</td>
<td>90</td>
<td>11.38</td>
<td>DB-CMI</td>
<td></td>
</tr>
<tr>
<td></td>
<td>M-600-90-1 Mortar</td>
<td>650</td>
<td>90</td>
<td>9.30</td>
<td>DB-CMI</td>
<td></td>
</tr>
<tr>
<td></td>
<td>M-700-90-1 Mortar</td>
<td>700</td>
<td>90</td>
<td>3.71</td>
<td>RF</td>
<td></td>
</tr>
<tr>
<td>G4</td>
<td>E-120-90-1 Epoxy</td>
<td>130</td>
<td>90</td>
<td>11.02</td>
<td>DB-A</td>
<td></td>
</tr>
<tr>
<td></td>
<td>E-150-90-1 Epoxy</td>
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<td>90</td>
<td>8.24</td>
<td>DB-A</td>
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<tr>
<td></td>
<td>E-200-90-1 Epoxy</td>
<td>200</td>
<td>90</td>
<td>7.81</td>
<td>DB-AFI</td>
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<tr>
<td></td>
<td>E-200-90-2 Epoxy</td>
<td>200</td>
<td>90</td>
<td>5.55</td>
<td>DB-AFI</td>
<td></td>
</tr>
</tbody>
</table>

¹ M-23-0-1:M = Mortar, 23= target temperature of specimen at failure, 0 = holding time at target temperature in minutes (0 = at room temperature), 1 = first specimen in sub-series.

² DB-CMI, debonding at concrete-mortar interface; DB-MFI, debonding at mortar-fibre interface; DF, delaminating between fibers; RF, rupture at FRP plate; DB-CAI, debonding at concrete-adhesive interface; DB-A, debonding at adhesive; DB-AFI, debonding at adhesive-fibre interface
Figure 4. Normalized ultimate strength at elevated temperatures

**Conclusions**

A description of the test set-up and results of a series of single-shear FRP-to-concrete joints using mortar as the bonding agent at elevated temperatures has been presented in this paper. The temperatures ranged from approximately 20 to 700℃. In the tests results, the failure modes of cement-based joints are more complex and more diversified, which could be caused by operation uncertainties flexibilities. However, there were no obvious reductions on the bond strength of FRP-to-concrete joints at such high temperatures compared with the room temperature. This work demonstrated that using mortar as bonding agent was a good approach to provide sufficient fire endurance for the externally-bonded FRP systems. Further research will be needed to establish the long-term effectiveness of the cement-based joints and the effectiveness of additives, which could increase the bond strength of the joint without changing its mineral based characteristics.

**ACKNOWLEDGMENTS**

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**REFERENCES**


Confinement
CYCLIC STRESS-STRAIN MODEL OF FRP-CONFINED CONCRETE FEATURING POST-PEAK STRAIN SOFTENING

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ABSTRACT

Many cyclic models for FRP (fibre reinforcement polymer) confined concrete have been developed based on the post-peak strain hardening type of stress-strain curves. However, the investigation of experiment conducted by authors recently indicated that the confinement level and unconfined concrete strength have a more pronounced effect on the strain softening type of stress-strain curve, including the unloading path and reloading path, compared with the strain hardening type of curves. By considering the influence of strain softening, more accurate expressions for the key shape factors of the cyclic curve regarding reloading stiffness, tangent unloading stiffness at zero stress, and plastic strain are proposed. These new expressions can be easily incorporated into the cyclic stress-strain model previously developed by authors for more accurate predictions especially when strain softening type of curve occurs. Good agreement between the predictions of the model with the test results represents the capability and accuracy of the proposed model.

KEYWORDS

Concrete, FRP, confinement, cyclic load, strain softening, stress-strain model.

INTRODUCTION

Many stress-strain models have been developed for FRP-confined concrete under cyclic loading in the existing literature. Shao et al. (2006) firstly developed a cyclic stress-strain model for FRP-confined concrete on basis of their test results. Lam and Teng (2009) questioned the accuracy of Shao’s model due to insufficient database and proposed an algebraic expression to model the cyclic response of FRP confined concrete. Wang et al. (2011, 2012) proposed two cyclic stress-strain models for FRP-confined reinforced concrete (RC) columns, one for square cross-sections and the other for circular cross sections based on their tests on large columns. Hany et al. (2015) conducted experimental tests and proposed a unified cyclic stress-strain model by adjusting the coefficients of the monotonic stress-strain model proposed by Lam and Teng (2013) as the envelope model, and adopted the same form of unloading model proposed by Wang et al. (2012). The authors also proposed a cyclic stress-strain model with a simple form and better accuracy of prediction (Li and Wu 2015).

Among these existing cyclic stress-strain models, only Hany et al.’s (2015) claimed that their model has the capacity to account for the post-peak softening behaviour for FRP-confined concrete cylinder. But, all the strain-softening test results collected by Hany et al. (2015) were obtained from non-circular columns in which the softening effect was caused by the shape of the column cross-section. No softening type of response has been obtained for columns with a circular cross-section and sufficiently low confinement. On the other hand, except the authors’ model (Li and Wu 2015), the other existing models do not take into account all the important factors. For example, in the residual plastic strain model which is an important parameter for unloading and reloading paths, the only factor considered is the unloading strain. Further two other important factors, the confinement and concrete strength, have not been taken into account. The author recently carried out a series of experiments for filling the test gap and reported some impartment findings (Li and Wu 2016). On basis of the authors’ test results, the above issues are properly addressed, and in this paper, a cyclic stress-strain model for FRP-confined concrete with both post peak strain-hardening and strain-softening is proposed with good accuracy.
TEST DATABASE

The proposed cyclic model in this paper was derived from the authors’ test result (Li and Wu 2016) consisting of strain-softening curves and others work (Ilki and Kumbasar 2002, Lam, Teng et al. 2006, Ozbakkaloglu and Akin 2011, Abbasnia, Ahmadi et al. 2012, Abbasnia, Hosseinpour et al. 2012, Wang, Wang et al. 2012, Hany, Hantouche et al. 2015) that mainly show strain hardening post peak behaviour. The model covers all important parameters in a unified manner, in terms of unconfined concrete strength, confinement rigidity, aspect ratio, corner radius etc.

PROPOSED MODEL

The full stress-strain model for concrete under cyclic compression is comprised of three main elements: envelope curve, unloading path and reloading path, as shown in Figure 1. The main parameters controlling the shape of these curves have been discussed above. According to the test results in authors’ recent study (Li and Wu 2016), the proposed cyclic model for FRP-confined concrete adaptable for both the post-peak strain-hardening and strain-softening range is proposed as follows.

Envelope model

Wei and Wu’s monotonic stress-strain model (Wei and Wu 2012) is employed herein to model the envelope curve. This model having a unified form for both post-peak strain-hardening and strain-softening is applicable to different shapes of cross-sections, which is given following:

\[
\begin{align*}
    f_c &= E_c \varepsilon_c + \frac{f_o - E_o \varepsilon_o}{\varepsilon_o^2} \varepsilon_c^2, \quad 0 \leq \varepsilon_c \leq \varepsilon_o \\
    f_c &= f_o + E_o (\varepsilon_o - \varepsilon_c), \quad \varepsilon_o \leq \varepsilon_c \leq \varepsilon_{co}
\end{align*}
\]

where, \(E_c\) and \(E_o\) are the first of second slope of monotonic response (Figure 1), respectively. \(f_o\) and \(\varepsilon_o\) are the transitional stress and corresponding strain, respectively. The details of the other parameters in Eq. (1) were provided by original paper (Wei and Wu 2012). However, by a regression analysis of the current test database, it is noted that Wei and Wu’s model (2012) underestimated the confinement effectiveness because only square and rectangular cross section were collected for modelling softening behaviour. Therefore, the coefficient of \(f_l/f_{co}\) in ultimate stress \((f_{cu})\) model is modified to 0.66 in this study, instead of 0.73, and it is determined by the following equation:

\[
\frac{f_{cu}}{f_{co}} = 0.5 + 2.7 \left( \frac{2r}{b} \right)^{0.4} \left( \frac{f_l}{f_{co}} \right)^{0.63} \left( \frac{h}{b} \right)^{-1}
\]

where \(f_{co}\) is concrete grade, \(2r/b\) is the corner radius, \(h/b\) is the aspect ratio. \(f_l\) is the confinement pressure at the fibre rapture point.
Unloading model

The proposed unloading model applied the same mathematical form, as adopted in the work of previous developed by authors (Li and Wu 2015). The nonlinear expressions (Eq. 3) of the unloading curve, describes well the shape of the unloading behaviour for FRP-confined concrete specimens having a post peak strain softening response. The details of the unloading model can be found in the original paper (Li and Wu 2015).

\[
f_r = E_{um,0} \left( \frac{\varepsilon_r}{\varepsilon_{pl}} \right)^m \left( \varepsilon_r - \varepsilon_{pl} \right)
\]

where \(\varepsilon_r\) is the plastic strain, and \(E_{um,0}\) is the tangent unloading stiffness at the plastic strain point. \(m\) is the shape factor that can be obtained by substituted unloading point (\(f_{um}\) and \(\varepsilon_{um}\)) into Eq. 3. The influence of the key parameters on the \(E_{um,0}\) has been shown in Figure 2, and the \(E_{um,0}\) decreases with the increase of unloading strain. In particular, this trend is more pronounced for specimens featuring a post peak softening behaviour such as C60P0.3C, C50P0.3C, and C50P0C. This observation reveals that the process of concrete damage and material softening is accelerated by the presence of strain softening.

By regression analysis of these test data, a new expression for the predicting parameter \(E_{um,0}\), based on the same mathematic expression with that proposed by authors (Li and Wu 2015), is developed as follows:

\[
E_{um,0} = 0.194 \left( \frac{f_{um}}{f_{30}} \right)^{0.276} \rho^{-0.015} \left( \frac{\varepsilon_{um}}{\varepsilon_{cop}} \right)^{-1.126}
\]

where \(f_{30}\) is the unconfined concrete strength of C30, and \(\varepsilon_{cop}\) is the axial strain at the compressive strength of unconfined concrete. \(\rho\) is the confinement stiffness ratio.

Reloading model

The proposed reloading model (Eq. 5) also adopted the same mathematical form with Li and Wu’s model (Li and Wu 2015), which was originally developed based on Richard and Abbott’s (1975) four parameters function. This expression could model the nonlinear transition zone between two approximately linear parts. Moreover, it also has the ability to predict the descending second part of reloading curve.

\[
f_r = \frac{(E_r - E_2)(\varepsilon_r - \varepsilon_{pl})}{f_r}\left(1 + (\frac{(E_r - E_2)(\varepsilon_r - \varepsilon_{pl})}{f_r})^{n}\right)^{-\frac{1}{n}}
\]

where \(E_r\) is the reloading stiffness, \(f_r\) is the reloading reference stress shown in Figure 1, and \(n\) is the transition parameter. According to the test results (Li and Wu 2016), it can be observed that parameter \(f_r\) displays a dropping trend with the negative \(E_2\). Hence, in the study, \(f_r\) was presented into two separate formulae according to the sign of \(E_2\), as show in Eq. 6. In which the first part was proposed for strain-hardening, and the second part was obtained from the regression analysis of strain-softening curves.
\[ \frac{f_{re}}{f_{co}} = 0.067 \frac{e_{un}}{e_{co}} + 0.733 \rho^{0.1} \quad E_2 \geq 0 \]
\[ \frac{f_{re}}{f_{co}} = -0.084 \frac{e_{un}}{e_{co}} + 1.536 \rho^{-0.122} \quad E_2 \geq 0 \]

Figure 3 shows the parameter effect on reloading stiffness \( E_{re} \), and it is clear that \( E_{re} \) is also highly dependent on unconfined concrete strength and confinement level for FRP-confined concrete specimens. Similar trends as those for \( E_{un} \) can be observed. It is noted that the rate of degradation is faster for concrete specimens with lower confinement (0.3-ply and 0.5-ply) compared with those with higher confinement (1-ply). This decrease trend is more pronounced for specimens featuring a post peak softening behavior. According to these discussion and regression analysis, the \( E_{re} \) expression is given following:

\[ \frac{E_{re}}{E_c} = \left( \frac{f_{re}}{f_{so}} \right)^{0.032} (\varepsilon)^{-0.409} - 0.317 \rho^{-0.064} \]

\[ \varepsilon = \begin{cases} \frac{e_{un}}{e_{co}}, & \text{when } e_{un} / e_{co} \leq 10 \\ 10, & \text{when } e_{un} / e_{co} > 10 \end{cases} \]

The performance of Eqs. 6 and 7 is shown in Figures 4 and 5, respectively.

The value of \( n \) is not as sensitive as other parameters. By combining the current test data and existing database, the model of \( n \) proposed herein is the same as the existing model developed by the authors (Li and Wu 2015):

\[ n = 2.61 \left( \frac{e_{un}}{e_{co}} \right) + 4.88 \]
Plastic strain model

Figure 6 shows that unconfined concrete strength and confinement rigidity are factors affecting the relationship between residual plastic strain and unloading strain, especially the effects caused by CFRP confinement level are more profound for specimens with a strain-softening behaviour. Through data analyses, the following model is obtained:
Concrete under Cyclic Loading

It can be seen from the figure that the proposed model matches both strain hardening and softening test results very well and hence performs very well.

EVALUATION OF PROPOSED MODEL

Figure 7 shows comparisons between predictions of the overall proposed model and the current test results. The unloading strains and reloading strains of the model curves match the test points and all other parts of the theoretical curves are predictions of the proposed model. It can be seen from the figure that the proposed model matches both strain hardening and softening test results very well and hence performs very well.

CONCLUSIONS

A complete cyclic compression stress-strain model was proposed by combining the envelope stress-strain model with the unloading and reloading path model. It is capable of predicting the ascending and descending post peak stress-strain response. For the FRP-confined concrete cylinders, the concrete strength and confinement stiffness ratio were found to influence the shape of the unloading curve, reloading curve and plastic strain. These effects are more profound for specimens exhibiting a post peak strain softening behaviour. Good agreement between the predictions of the model with the test results of this paper represents the capability and accuracy of the proposed model.

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REFERENCES


FRICTION EFFECT OF CONCRETE SPECIMENS UNDER UNIFORM PASSIVE CONFINEMENT IN TRUE-TRIAXIAL COMPRESSION TEST

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ABSTRACT

Fibre-reinforced polymer (FRP) has been extensively used in the retrofit of concrete structure, especially for concrete columns. FRP provides uniform passive confinement for circular columns under axial compression while for non-circular columns and columns under compression with eccentricity, the confinement is non-uniform. True-triaxial experiment is essential to obtain the mechanical properties of concrete under non-uniform confinement. In this paper, the triaxial behaviour of concrete cubes under uniform passive confinement was studied using a new triaxial setup. The results were compared with those of FRP-confined concrete cylinders which have same strength and lateral stiffness ratio. End-friction influence on both fresh concrete cubes and passively confined cubes was analysed. The axial stress-strain behaviour of cubes was modelled including the friction effect. Then the curves of cubes with and without friction effect were compared with those of cylinders. At last, the lateral strain-axial strain curves of cubes and cylinders were analysed. It comes to conclusion that, excluding the friction effect, the stress-strain curve of cubes fit well with that of cylinders. Further, the new testing equipment is ready to provide non-uniform passive confinement for concrete cubes, which can assist the numerical modelling of FRP-confined elliptical or rectangular columns for the sake of engineering practice.

KEYWORDS

Concrete, friction effect, true-triaxial compression, uniform passive confinement.

INTRODUCTION

It is well known that by providing lateral confinement for concrete elements the ultimate axial compressive strength increases considerably. That’s the mechanism of fibre-reinforced polymer (FRP) confined concrete. FRP provides passive confinement, which due to the elongation imposed on it because of expansion of concrete, induces confinement for column. For circular columns, it is commonly accepted that the passive confinement is uniform while for non-circular columns and columns under eccentric compression, the passive confinement is non-uniform. The corresponding mechanical behaviour of concrete is usually studied by wrapping FRP jacket around concrete cylinders (e.g. Xiao and Wu 2003). Then the stress-strain relationship is established based on the experimental results (e.g. Lam and Teng 2003), in which the lateral strain in the middle and average axial strain along the height are commonly adopted as the lateral and axial strain in the constitutive model. Through digital measurement, it can detect the non-uniform strain field in longitudinal direction (Figure 1). The non-uniform deformation can be attributed to the end-friction effect on the column. The additional confinement will exist at 1/3 of the height at each end zone (Figure 1a). Strictly speaking, the recorded stress-strain curves still reflect the structural behavior rather than the material mechanism. Therefore, it needs a more efficient testing method to study the mechanism of concrete under passive confinement. A novel triaxial equipment has been invented by the authors (Jiang et al. 2015) to provide passive confinement for concrete cubes. Regarding the true-triaxial testing, the specimens have different dimension to the standard cylinder. The friction effect of cubes, regarding the stress boundary condition at loading end, needs to be clarified to obtain the inherent mechanical properties of concrete.

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EXPERIMENTAL PROGRAM

The adopted new triaxial equipment consists of two parts, the vertical loading part and the passive loading part. It was tailored for 100mmx100mmx100mm cubes. The vertical force is applied with the universal testing machine. The passive loading apparatus is composed by the steel box and GFRP bars. The details about the apparatus can be found in Jiang et al. (2015). When the cube is subjected to vertical force, its lateral deformation will initiate the elongation of bars which will generate confinement pressure linearly proportional to the expansion of concrete. To reduce friction on the cube surface, layers of PTFE with an intermediate layer of grease were placed between each surface of the cube and the steel loading plate.

In the experiment, concrete cubes were divided into three groups. Their 28-day strengths were 25.4MPa, 36MPa and 44MPa, respectively. For each group of concrete specimens, uniform passive confinement was provided with four different loading schemes, evaluated by lateral stiffness ratio $\rho=d(f_{f(c)})/d\delta$. This was achieved by using GFRP bars of four different diameters. Table 1 listed the details of specimens. Specimen ID for cubes shows four different loading schemes, evaluated by lateral stiffness ratio $\rho$. The deformation and load data were collected through loading. The measurement scheme is detailed in Jiang et al. (2015).

<table>
<thead>
<tr>
<th>Spec. ID</th>
<th>$\rho$</th>
<th>Spec. ID</th>
<th>$\rho$</th>
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<td>13.8</td>
</tr>
<tr>
<td>C-C-20</td>
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<td>25.0</td>
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<tr>
<td>D-D-20</td>
<td>39.1</td>
<td>D-D-30</td>
<td>27.6</td>
<td>D-D-40</td>
<td>23.0</td>
</tr>
</tbody>
</table>

STRESS-STRAIN BEHAVIOR OF CONCRETE CUBE UNDER PASSIVE CONFINEMENT

The specimens under biaxial uniform confinement were characterized by bi-linear strain-hardening (Figures 2a-2c), which have the same pattern as FRP-confined cylinders. Similarly, the second linear slope decreased when $\rho$ was decreased (from D-D to A-A). The stress-strain curve can be expressed by the piece-wise curve (Eq.1), composed by a parabolic curve followed by a linear curve, as proposed by Lam and Teng (2003) for FRP-confined cylinders. The intersection point $(\varepsilon_0, f_0)$ is termed as the transition state point. The stress-strain curve bifurcates at the transition state point where the passive confinement takes effect.

$$
\sigma_\varepsilon = \begin{cases} 
E_\varepsilon \varepsilon_0 - \frac{(E_\varepsilon - E_0)^2}{4f_0} & (0 \leq \varepsilon_0 \leq \varepsilon_0) \\
\frac{f_0 + E_\varepsilon \varepsilon_0}{E_\varepsilon} & (\varepsilon_0 \leq \varepsilon_0 \leq \varepsilon_0) 
\end{cases}
$$

(1)

$$
\varepsilon_0 = \frac{2f_0}{E_\varepsilon - E_0}; f_0 = (f_0 - 9)^{1/2}; E_0 = 4.9f_0^{0.675}
$$

(1a,b,c)

where $E_\varepsilon$ is the Young’s modulus, $E_0$ is the slope of second line; $f_0$ is the intersection of second line with the stress axis; $E_2$ and $f_0$ are regressed from the test data.
FRICTION EFFECT ON MECHANICAL PROPERTIES OF PASSIVELY CONFINED CONCRETE CUBE

Due to different Poisson’s ratios between the steel loading plate and the concrete, the non-consistence in expansion between two materials generates the friction between the two surfaces under axial compression. The end friction is the major reason for the variance in concrete specimens with different slender ratio. In order to obtain the inherent characteristic of material, end friction reduction is the basic approach to eliminate the friction effect.

End Friction Reduction for Fresh Concrete Cube

With respect to the dimension of cube, the end-friction can be equivalent to lateral confinement exerted along overall height (Figure 3a). The additional restraint in the specimen end was expected to increase the concrete strength. To eliminate the end-restrain effect, the friction-reduction pad has been commonly adopted in triaxial testing (Van Mier et al. 1997). Herein PTFE laminate (0.05 mm/ply) was adopted to investigate the appropriate number of layers which ranged from 1 to 3. For those with more than one-ply PTFE laminate, the specimens were divided into two groups: with or without grease. The properties of the specimens are detailed in Table 2, together with their test results.

With the increase of the PTFE ply number, the concrete showed a substantial reduction of its peak strength (Figure 3b). In other words, the increase of PTFE layers can achieve smaller friction coefficient between two surfaces. The reduction percentage remained constant when the layer number was larger than two, while the side-wedge peeling could still be observed and the end friction still existed. With additional grease between the laminate layers, the pads of the 2- and 3-ply PTFE laminate can have smaller friction coefficient to slide. The peak strength further decreased to half of the cube strength without a friction-reduction pad. The failure pattern was characterized by

Table 2 Equivalent confinement with respect to the end-friction effect

<table>
<thead>
<tr>
<th>Spec. ID</th>
<th>PTFE layer</th>
<th>Grease</th>
<th>f′c (MPa)</th>
<th>Triaxial failure criterion function</th>
<th>f_{l,eq}/f_{c0}</th>
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<tr>
<td>a0</td>
<td>0</td>
<td>No</td>
<td>40.3</td>
<td></td>
<td>0.19</td>
</tr>
<tr>
<td>a1</td>
<td>1</td>
<td>No</td>
<td>34</td>
<td></td>
<td>0.11</td>
</tr>
<tr>
<td>a2A</td>
<td>2</td>
<td>No</td>
<td>25</td>
<td>( f_{l,c} = 1.37 (f_{c0}/f_{x})^{0.86} )</td>
<td>0.02</td>
</tr>
<tr>
<td>a3A</td>
<td>3</td>
<td>Yes</td>
<td>25</td>
<td></td>
<td>0.02</td>
</tr>
<tr>
<td>a2B</td>
<td>2</td>
<td>Yes</td>
<td>22</td>
<td>(Wang et al. 1987)</td>
<td>0</td>
</tr>
<tr>
<td>a3B</td>
<td>3</td>
<td>Yes</td>
<td>19.9</td>
<td></td>
<td>0</td>
</tr>
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</table>
splitting cracking. Accordingly, specimen a2B and a3B were regarded as free of end restrain. By comparison, those with end friction effect experience lateral confinement (Figure 3a) and exhibit similar phenomenon to the cube under constant confinement. The confinement pressure can be derived by introducing the failure criterion function from Wang et al. (1987), and deemed as the equivalent confinement, \( f_{lt} \) (Table 2). It can be found that the direct attachment between steel plate and concrete can exert lateral confinement pressure equivalent to as much as 20% of the true peak strength. Accordingly, the 2-ply PTFE with an intermediate layer of grease was adopted as the friction-reduction pad for the following triaxial compression tests.

**End Friction Reduction for Passively Confined Concrete Cube**

With additional lateral passive confinement, the cube has two major features different from fresh cube, that is, the delayed dilation and continuous increase in axial loading bearing. As reported in Van Mier et al. (1997), the Teflon friction reduction pad has stick-slip behavior. The free of friction force will be stabilized after certain sliding. In practice, the free of restrain will be fully generated at post-peak for fresh concrete due to fast development in lateral expansion. For passively confined cube, the severe lateral dilation is restrained by progressive confinement, which will postpone the state of free of restrain. Moreover, the larger axial loading forces the grease to be squeezed out of Teflon pad. It can be observed that the oil spots scattered on the loading surface of tested specimen which reflects the uneven contact surface. The micro-cracking on loading surface may rough the pad under high level of compression, which will increase the friction coefficient between interlayer sliding.

The axial stress at transition point \( (\epsilon_{lt}, f_{lt}) \) is adopted as the reference point to evaluate the additional restraint caused by end friction. Since concrete experienced minor damage before it reaches the transition state, it can consider non-path dependence between passive and active confinement (Xiong et al. 2016). By introducing the failure criterion function obtained from traditional triaxial tests (Wang et al. 1987), the lateral confinement \( f_{lt} \) required to achieve \( f_{lt} \) is obtained. The resulting values of \( f_{lt} \) for current specimens exhibit much larger than the lateral confinement \( f_{lt} \) exerted by the lateral loading apparatus corresponding to the transition point (Figure 4). The difference in the experiment is attributed to the additional restraint because of end friction. The effect of end friction is related to fresh concrete strength.

![Figure 4 Comparison of \( f_{lt} \) and \( f_{lt} \)](image)

**COMPARISON BETWEEN PASSIVELY CONFINED CUBE AND CYLINDER**

<table>
<thead>
<tr>
<th>Group</th>
<th>Group ( f_{c0} )</th>
<th>Group ( \rho )</th>
<th>Specimen ID</th>
<th>Source</th>
<th>Type</th>
<th>( f_{c0} ) (MPa)</th>
<th>( \rho )</th>
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<tr>
<td>I</td>
<td>25</td>
<td>20</td>
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<td>Cylinder</td>
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<td>Cube</td>
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<td>II</td>
<td>36</td>
<td>15</td>
<td>C40CF1</td>
<td>Wu and Jiang 2013</td>
<td>Cylinder</td>
<td>36.67</td>
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<td>36</td>
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<td>Experiment</td>
<td>Cube</td>
<td>36</td>
<td>27.6</td>
</tr>
<tr>
<td>IV</td>
<td>45</td>
<td>20</td>
<td>C47.6CF3</td>
<td>Jiang and Teng 2007</td>
<td>Cylinder</td>
<td>47.6</td>
<td>22.85</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>D-D-40</td>
<td>Experiment</td>
<td>Cube</td>
<td>44</td>
<td>23</td>
</tr>
</tbody>
</table>

Previous research has been done on 150mm×300mm concrete cylinders (Jiang and Teng 2007; Wu and Jiang 2013). To exclude the effect from unconfined concrete strength and confinement stiffness, the compared cylinder specimens were selected based on the same value of concrete strength \( f_{c0} \) and lateral stiffness ratio \( \rho \) (Table 3). Herein, the end friction effect on the stress is excluded from the testing results. Axial stress at transition point \( f_{lt} \) is substituted with the axial stress \( f_{lt} \) calculated from triaxial failure criterion function setting the lateral confinement as \( f_{lt} \). Then the curves of cubes with and without friction effect are compared with those of cylinders in the same group in Figure 5. Figure 6 illustrates the comparison of lateral strain vs. axial strain curves. As can be seen, the axial stress-strain curves have close match between cylinders and cubes, but with minor variation without clear trend. The same phenomenon can be observed in lateral strain vs. axial strain curves. However, it
can be found the axial strain in cube has the same difference trend at the same axial stress and lateral strain, to compare with cylinder. That demonstrates that the axial strain detected in the cube reflect the localized value in the cylinder. And since the axial strain data scatters along the longitudinal direction (Bisby and Take 2009, Figure 1b), it also results in the comparison having no clear trend.

Figure 5 Comparison of axial stress-axial strain curves between cylinders and cubes

Figure 6 Comparison of lateral strain-axial strain curves between cylinders and cubes
CONCLUSIONS

In this paper, the mechanism of concrete under passive confinement was studied with triaxial experiment on concrete cubes using a new triaxial equipment. The friction effect of cubes was clarified to obtain the inherent mechanical properties of concrete. Conclusions have been drawn as follows:
(a) Axial stress-axial strain curve of cubes under biaxial uniform confinement were characterized by bi-linear strain-hardening, which have the same pattern as cylinders. The second linear slope decreased when lateral stiffness ratio was decreased. The stress-strain curve was expressed by the piece-wise curve, composed by a parabolic curve followed by a linear curve.
(b) End friction is the major reason for the variance in concrete cubes and cylinders. For fresh concrete cube, with increase of PTFE ply number, concrete showed a substantial reduction of its peak strength. With grease between 2-ply PTFE laminate, the peak strength decreased to half of the cube strength without a friction-reduction pad. The direct attachment between steel plate and concrete can exert lateral confinement pressure equivalent to as much as 20% of the true peak strength.
(c) End friction effect on passively confined concrete was quantified. It results additional confinement which is related to the unconfined concrete strength. When friction effect on axial stress is excluded from original testing data, the axial stress-axial strain curves of cubes have minor difference to those of cylinders. The corrected cubic results reflect the localized condition in FRP confined cylinders.

ACKNOWLEDGEMENTS

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CONCRETE CUBE UNDER MULTI-AXIAL PASSIVE CONFINEMENT

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ABSTRACT

A novel equipment is introduced which is able to conduct tri-axial and bi-axial passive confined tests on concrete cubes. Lateral stiffness ratio is provided by which the level of confinement can be adjusted for each lateral direction. An experimental set-up is proposed which is used for calibration of the measurement system in lateral directions. The friction between loading platens and concrete has been released so that the test results are validated in comparison to typical circular FRP confined columns. Comparison of preliminary tests with finite element results reveals that the proposed experiment is able to verify the accuracy of models which are developed for three-dimensional modelling of non-uniform FRP confined concrete columns.

KEYWORDS

FRP, tri-axial, bi-axial, load path dependent, concrete cube, dilation properties, passive confinement.

INTRODUCTION

Fiber reinforced polymers (FRPs) are used to enhance the strength and ductility of concrete columns. Numerous empirical and semi-analytical models have been developed for FRP confined concrete columns (Jiang and Teng 2007; Wei and Wu 2012). Also, plasticity based material models are used with finite element methods (FEM) to capture the strength and deformational behavior of such structures (Karabinis et al. 2008; Yu et al. 2010a; Yu et al. 2010b). For FEM, users need to adjust the parameters of constitutive material models. This adjustment is done by using tri-axial concrete tests (Jiang and Wu 2014; Rousakis et al. 2007; Teng et al. 2015). All existing tri-axial concrete cube tests are for active confinement. Concrete is load path dependent material and recently it was shown that behavior of concrete under passive confinement is different from that in active confinement (Lim and Ozbakkaloglu 2014; Ozbakkaloglu et al. 2016). Also, the lateral deformations of concrete in tri-axial tests are required for calibration of material models. Existing active concrete cube tests can hardly get the lateral deformations after peak stress.

For addressing the above issues, this study introduces a tri-axial testing equipment for conducting multi-axial passive confined tests on concrete cubes. The confinement level in each lateral direction is adjustable so that different non-uniform passive confinement i.e. tri-axial and bi-axial tests can be conducted. The preliminary results for monotonic tests are illustrated. Finally, FEM is used by concrete damage-plasticity material model (Yu et al. 2010b) to compare experiments with numerical results. The proposed test is able to verify the accuracy of models which are developed for three-dimensional modeling of FRP confined concrete columns.

EQUIPMENT DESIGN

The second author (Jiang and Wu 2014) for the first time proposed the design of tri-axial passive testing equipment as shown in Figure 1. A concrete cube is confined by metal plates and in lateral directions clamped elastic bars are used to imitate the passive confinements. The axial force is applied in the direction normal to the paper and concrete expands in lateral directions so that lateral confinements are activated. This idea for the first time could be implemented at Tongji University (Jiang et al. 2015) by means of FRP bars. However, as shown in Figure 1b the clamping system is very large due to the problem of threading FRP bars. In present study as shown in Figure 1a, Aluminium bars are adopted for elastic bars and standard steel nuts are used for clamping. Also, Aluminium is used for confining plates to minimize the weight of plates. The size of the concrete cube is 150 by 150 mm.

In typical FRP confined columns the lateral stiffness ratio \( \rho \) is a function of FRP stiffness \( E_{frp} \), FRP thickness \( t_{frp} \), unconfined concrete strength \( f_{cu} \) and diameter of column \( D \) (Jiang and Wu 2012):
\[ \rho = \frac{2E_{\text{bar}}A_{\text{bar}}}{Df_{\text{co}}} \]  

(1)

(a) Proposed design  
(b) First implementation (Jiang et al. 2015)

Figure 1 Equipment design

In present equipment by using equilibrium and compatibility between concrete cube and confining plates, the following lateral stiffness ratio is obtained as a function of elastic bar stiffness \( E_b \), length of bars \( L_b \), cross-sectional area of bars \( A_b \) and cross-sectional area of concrete \( A_c \):

\[ \rho = \frac{4E_b}{f_{\text{co}}} \left( \frac{A_b}{A_c} \right) \left( \frac{L_c}{L_b} \right) \]  

(2)

The coefficient of four is for having four bars in each lateral direction. Therefore in each lateral direction by equating Eqs. (1) and (2) the thickness and length of elastic bars are calculated.

CALIBRATION

In real tests, Linear Variable Differential Transducers (LVDTs) are used to capture lateral displacements. In Figure 2, a couple of confining plates is shown. Two LVDTs named as LVDT-S are located at sides, the same as real test measurement system to measure concrete lateral deformations. However, the confining plates will bend due to concrete expansion. The designed plates should be rigid enough in order to capture correct lateral displacement of the concrete cube. Plates are designed and their flexural rigidity is checked by FEM modelling. Finally, a calibration procedure is designed as in Figure 2. A hydraulic jack is located between the plates. All system is located on a surface covered with greasy Teflon in order to let the plates move freely. Figure 2 shows the top view. The length of jack plus other components located between plates is almost equal to the length of the concrete cube. The hydraulic jack pushes the plates while they are clamped with Aluminum bars (shown with dashed lines). The displacements are measured through the couple of LVDT-S at sides as well as by the couple of LVDT-C at center. The overall measurement at the central LVDTs should be equal to that for the side LVDTs. If this condition is not met then the rigidity of plates should be increased e.g. by changing the thickness of plates.

In real tests, the lateral displacement measured by the couple of LVDT-S is not equal to concrete lateral deformations. It includes extra deformations related to other components i.e. Teflon and confining plates. To measure the extra deformations, as shown in Figure 2, another type of calibration is conducted. This time, the couple of LVDT-M, the couple of LVDT-S and strain gauges are used for measurements. The geometry and stiffness of steel cubes are known. Therefore, lateral pressure is calculated by means of strain gauges. Also, the value measured by the couple of LVDT-M is subtracted from that measured by LVDT-S to calculate extra part of measured lateral displacements. As a result, the extra lateral displacement is obtained as a function of lateral pressure. Such a calibration test should be repeated for all type of elastic bar diameters which are used in real tests. Calibration in the axial direction is referred to Mohammadi and Wu (2016) for the sake of brevity.
VALIDATION

When the level of confinements in lateral directions are equal, then the cube test is the same as a typical circular FRP confined column. However, the friction between the concrete cube and loading platens should be released. Trial tests of Mohammadi and Wu (2016) revealed that three layers of Teflon sheets should be used at top and bottom faces of the concrete cube and a single Teflon layer is needed for other faces to release the friction properly. Adopted Teflon thickness is 0.05 mm. The final validation test is shown in Figure 3 in which cylindrical column tests are compared to present tests for one ply CFRP confinement. The thickness and stiffness of FRP are 0.165 mm and 245 GPa respectively.

RESULTS

Preliminary monotonic tests are depicted in Figure 4 in which the major confinement level is one ply of CFRP (thickness per ply=0.165 mm, stiffness=245 GPa). The minor confinement level is changing with half and zero plies of CFRP. The thickness and length of Aluminium bars are calculated by equating Eqs.(1) and (2). Evidently, stress-strain of a non-uniform confined cube is similar to a circular FRP confined column. But the ultimate strength
decreases when the minor level of confinement is decreased. Also, each lateral direction dilates with a constant different rate. However, the bi-axial stress-strain behaviour of concrete is totally different and the confined direction does not dilate after peak strength. This type of behaviour may happen in non-uniform FRP confined columns near the side of cross sections, especially in rectangular confined columns.

![Diagram](image-url)

**Figure 4** Tri-axial and bi-axial passive confined compression tests \( f_{c'} = 25.63 \text{MPa} \) (a) 1 Plies-1 Plies CFRP (b) 1 Plies-0.5 Plies CFRP (c) 1 Plies-0 Plies CFRP (d) 0.5 Plies-0 Plies CFRP

**DISCUSSION**

In modelling of non-uniform FRP confined columns e.g. square columns the elements near the rounded corners sustain uniform confinement. Also, the elements at sides of the cross section may sustain bi-axial or triaxial state of stress. The present experiments can provide the experimental data for verification of models which are developed for three-dimensional modelling of FRP confined concrete columns. A proper constitutive material model should be able to capture the stress-strain behaviour as well as dilation behaviour of bi-axial and tri-axial passive confined cube tests. Besides this, the material model should be able to capture the global strength and deformation of a non-uniform FRP confined columns. In this regard, for three-dimensional modelling of FRP confined columns the advanced model developed by Yu et al. (2010b) which is able to model the behaviour of non-uniform FRP confined columns is adopted. That model utilizes analysis oriented model of Teng et al. (2007) to calibrate the Concrete Damage-Plasticity Model developed by Lubliner et al. (1989) and implemented in ABAQUS. The accuracy of that model has been clarified by FRP confined columns and here the present experiment provides the only way for verifying the accuracy of that model (and any other model) for capturing the behaviour of concrete elements under non-uniform passive confinements.

To this aim, the concrete material is modelled with Concrete Damage Plasticity the same as Yu et al. (2010) model. However, the required analysis oriented model is replaced with the model of Jiang and Teng (2007) which is superior for capturing the softening behaviour compared to the old model. A single concrete element is modelled and the passive confinement is applied on each face. The results are shown in Figure 5. As shown in Figure 5a-b whenever the dilation behaviour is captured successfully then the stress-strain is captured very close to experiments.
However, as shown in Figure 5c, FEM cannot capture the dilation properties successful enough for softening cases then the strength behaviour is not captured very well.

Figure 5 Comparison of tests with concrete damage plasticity developed by Yu et al. (2010) : (a) 1 Plies-1 Plies CFRP (b) 1 Plies-0.5 Plies CFRP (c) 1 Plies-0 Plies CFRP (d) 0.5 Plies-0 Plies CFRP

The present test method is able to provide the experimental data for verification as well as modification of material models which are developed for three-dimensional modelling of non-uniform FRP confined columns. Establishing a proper dilation properties such as the one proposed by Teng et al. (2007) is essential for developing analysis oriented models and for developing calibration procedures of plasticity based material models. In this regard, for dilation properties of non-uniform confined columns, the lateral strain captured around the perimeter of columns is variable and dependent on how to attach strain gauges on the surface of the column (Wang and Wu 2008). The present test uses LVDTs for capturing the dilation under non-uniform passive confinement. The proposed measurement system can provide accurate dilation properties.

CONCLUSION

A new multi-axial testing equipment was introduced which is able to conduct biaxial and tri-axial passive confined compression tests. The calibration procedure for lateral strain measurement system was elaborated. Therefore the system can be replicated by others. The friction on concrete cube faces could be released. Typical FRP confined concrete columns were used to validate the stress-strain and the dilation properties captured by presented equipment. Comparison of tests with FEM results shows that the present tests can provide a simple way for verifying the accuracy of any material model developed for three-dimensional modelling of FRP confined concrete columns.
ACKNOWLEDGMENTS

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AXIAL STRENGTH OF CFRP REPAIRED RC CIRCULAR COLUMNS DAMAGED BY FIRE

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ABSTRACT
Reinforced concrete structures have good fire resistance due to the inherited properties of concrete. Depending on the intensity and duration of fire, concrete properties deteriorate at different rates. Several researchers have documented the compressive properties of fire damaged concrete. Carbon fiber reinforced polymer (CFRP) composites have been successfully used for strengthening of circular column. Transverse wrapping of circular column could enhance confinement, shear resistance, enhance flexural ductility, and increase axial strength. CFRP wrapping increases the both axial strength and ductility of circular concrete columns, including the columns with low concrete compressive strength damaged by fire. This paper presents analytical investigation into the axial strength of fire damaged concrete columns strengthened with transverse CFRP wrapping. The results in this research suggest that CFRP wrapping should increase the axial strength of fire damaged circular concrete columns.

KEYWORDS
Axial strength, circular concrete column, fire damaged concrete, CFRP wrapping.

INTRODUCTION
Fire damage is one of the leading causes of structural damage around the world. With increasing population densities and larger buildings being built, it is safe to say that concrete structural-fires will be occurring at an increasing rate in the future unless methods to stop them are implemented.

The changes that occur to fire damaged concrete structures depend on the temperature level achieved, exposure time, and the cooling conditions. Concrete is a versatile material and when properly specified, is comparatively fire proof compared to most building materials because of its non-flammability and thermal insulation properties. However, severe temperatures and prolonged exposure to a high temperature fire results in reduction in both the compressive and tensile strength of concrete. The thermal changes cause the aggregate in the concrete to breakdown by means of cracking, which will lead to phenomenon called disaggregation. This is when the bond between the concrete paste and the aggregate is lost. Other phenomenon’s that occur to concrete when exposed to fire is explosive spalling, sloughing off, fine cracks, and wide cracks. Most fine cracks are confined to the surface where wide cracks near supports may mean a loss of anchorage or yielding of reinforcement bars. A number of changes will occur when concrete is exposed to high temperatures and they are all dependent on the temperature of the fire at that time. Concrete strength is affected by the variation of temperature, rate of heating, duration of heating, imposed load, type and size of aggregate, percentage of cement paste, and the water cement ratio. High temperatures will result in concrete compressive and tensile strength reduction, disaggregation, spalling, and cracking.

Carbon fiber reinforced polymer (CFRP) composites have been used for structural strengthening in the United States for almost 25 years. During this period, acceptance of CFRP composites as a mainstream construction material has grown, and so has the number of completed FRP strengthening projects. As a result, the use of CFRP for strengthening and rehabilitation is gaining more popularity among design professionals over conventional strengthening techniques.

CFRP strengthening of fire damaged concrete structures can involve complex evaluation, design, and detailing processes, requiring a good understanding of the existing structural conditions along with the materials used to repair the structure prior to CFRP installation. In the past few decades, research has demonstrated that for a compression member, the use of CFRP wrapping can result in an increase in load-carrying capacity and ductility.
In terms of the repair of post-heated concrete members, only a few studies have considered the use of CFRP fabrics for subsequent strengthening (Saafi and Romine, 2002, Cleary et al., 2003, Bisby et al., 2011, Yaqub and Bailey, 2011, Al-Kamaki and Al-Mahaidi, 2013). To date, several testing has been undertaken on RC-loaded columns by using electrical furnaces. However, these studies didn’t cover the analytical investigation of fire damaged concrete structures repaired with CFRP wrapping.

ANALYTICAL INVESTIGATIONS

Effects of elevated temperatures on the concrete properties

Concrete is regarded as a heat resistance material compared with other materials because of the low thermal conductivity of concrete at high temperatures. The thermal properties of concrete are highly dependent on the type of aggregate in the concrete. The certain aspects of concrete is affected by different aggregate types when exposed to high temperatures. These high temperatures caused by fire may create internal shear stresses due to the differences in thermal properties between the cement paste and the aggregates. Several concrete properties like the compressive strength of concrete at high temperatures varies according to the type of aggregate, cement to aggregate ration, and the degree of loading, among some other factors.

The temperature level achieved, exposure time, and the cooling conditions together controls the changes that occur to concrete. Concrete is a versatile material and when properly specified, is practically considered “fire proof” compared to most building materials because of its non-flammability and thermal insulation properties. However, severe temperatures and prolonged exposure to a high temperature fire will definitely results in decreases in both the compressive and tensile strength in concrete. The thermal changes results the aggregate in the concrete to breakdown by means of cracking, which will lead to phenomenon called disaggregation. This is when the bond between the concrete paste and the aggregate is lost. Other phenomenon like explosive spalling, sloughing off, fine cracks, and wide cracks will occur to concrete when exposed to high temperature. Most fine cracks are confined to the surface where wide cracks near supports may mean a loss of anchorage or yielding of reinforcement bars. These cracks are very import when examining a member after exposed to fire. A professional must be able to understand and evaluate what the concrete member experienced throughout fire based on visual examinations let alone the tests taken.

A series of changes will occur when concrete is exposed to high temperatures. These changes are dependent on the temperature of the fire at that time. Concrete strength is affected by the variation of temperature, rate of heating, duration of heating, imposed load, type and size of aggregate, percentage of cement paste, and the water cement ratio. High temperatures will lead to the reduction of concrete compressive and tensile strength, disaggregation, spalling, and cracking.

Concrete remaining strength after fire

The remaining strength of concrete after fire damage cannot be decided by simply ways of evaluation or calculation. Normally, when concrete rises above 100°C, moisture is driven out and the cement paste will begin to dehydrate. The aggregate particles will start to expand generating large internal strains. Micro cracking may begin, however, the integrity of the concrete is not really affected. When the temperatures exceed 300 °C, compressive strength will be threatened. Tensile strength is reduced immediately and continues to decrease as temperatures increase. Above 350 °C, some aggregates will dehydrate and begin to break off, particularly flint gravels. Some aggregates can withstand much higher temperatures such as crushed granite. As temperature increases other phase transformation occurs such as successive dissociation of calcium hydroxide and calcium carbonate. This may happen before the melting point of 1200 °C. A temperature line has been made to help better understand what a reinforced concrete member will generally go through as a fire increases in temperature throughout the burning phase (Figure 1). The concrete color will change throughout the rise in temperature and usually depends on aggregate type.

Because concrete has such great thermal properties, concrete itself is considered as a shield against the high temperatures. Generally, a cross section of concrete column with various temperature gradients throughout has areas of varying strength due to those temperature gradients. Each of these differential areas of the cross section contributes strength to the overall strength of the column. For ease of design and analysis, several conservative methods have been proposed to deal with temperature variations across the concrete section. The 500°C isotherm method and the 400/600°C isotherm method are two of those methods.
The first method called 500°C isotherm method considers that any portion of the cross section above 500°C has zero strength while only the part of section below 500°C contributes strength. Another method, the 400/600°C isotherm method uses two ways instead of one to predict the strength of the column. The method assumes that concrete sections below 400°C acquire 90% strength, those between 400°C and 600°C acquire 70% strength, and sections above 600°C have no contribution at all.

These are all approximate estimation methods because the actual strength of concrete decreases continuously from ambient temperature. Figure 2 shows the compressive strength of siliceous aggregate concrete. The compressive strength started decrease from about 100°C and the remaining strength decreased to about 20% of its original strength when the temperature reached about 700°C. The isotherm methods try to approximate the strength curve while still allowing for ease of calculations. The red line in Figure 2 is the strength assumption of the 500°C isotherm line, and the blue line is the strength assumption of the 400/600°C isotherm method. Each of these methods assumes that the cross-section remains intact throughout the entire heating and cooling process (Buchanan, 2002) (Society of Fire Protection Engineers, 2008).

Figure 3 below shows how a concrete column with diameter of 24in. will resist a typical fire that lasts for 2 hours. It also shows the strength of the same column in percentages after 2 hours. Based on the temperatures, the
remaining strength of the column would most likely be efficient enough to support the load that it was designed for. (Lie, T. T. (1992))

Figure 3 Concrete resistance to fire (Lie, T. T. (1992))

A reduction coefficient should be considered when measuring the remaining strength of the fire-damaged concrete after the cooling. The reduction coefficient is quite different according to different temperature and is also affected by the different cooling methods and type of the concrete. Table 1 shows the result of one experiment applied by W. Kong fan. It shows the performance of concrete cooled down from high temperature.

<table>
<thead>
<tr>
<th>Temperature</th>
<th>20°C</th>
<th>200°C</th>
<th>300°C</th>
<th>400°C</th>
<th>500°C</th>
<th>600°C</th>
<th>700°C</th>
<th>800°C</th>
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<tbody>
<tr>
<td>Natural cooling</td>
<td>1.00</td>
<td>1.00</td>
<td>0.82</td>
<td>0.76</td>
<td>0.60</td>
<td>0.60</td>
<td>0.50</td>
<td>0.25</td>
</tr>
<tr>
<td>Water cooling</td>
<td>1.00</td>
<td>1.00</td>
<td>0.74</td>
<td>0.63</td>
<td>0.52</td>
<td>0.43</td>
<td>0.29</td>
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</tr>
<tr>
<td>During fire</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>0.90</td>
<td>0.75</td>
<td>0.62</td>
<td>0.50</td>
<td>0.25</td>
</tr>
</tbody>
</table>

The Method of Applying CFRP to Concrete Column

Material Property

Table 2 below shows the properties of the CFRP materials used in this investigation.

| Thickness per layer t<sub>f</sub> | 0.0113in. |
| Rupture strain ε<sub>fu</sub> | 0.0167in./in. |
| Modulus of elasticity E<sub>f</sub> | 33000ksi |

Calculation of the strength gained from CFRP wrapping

CFRP systems can be used to enhance the axial compression strength of a concrete member by providing confinement with a CFRP jacket. Confining a concrete member is accomplished by orienting the fibers transverse to the longitudinal axis of the member. In this orientation, the transverse or hoop fibers are similar to conventional spiral or tie reinforcing steel. Any contribution of longitudinally aligned fibers to the axial compression strength of a concrete member should be neglected. (ACI.2R_08). In this study, Lie, T. T. (1992)’s evaluation method combined with the reduction coefficient was adopted, which calculates the remaining strength of concrete column by dividing its cross section into several layers as shown in Figure 4. Different diameter and type of concrete and different condition of fire may lead to quite different result. We just take the temperature to be around 500°C and other conditions to be mentioned above to see the result. Assume the diameter of the column to be 100 cm x 100 cm.
The axial compressive strength of a non-slender, normal weight concrete member confined with an FRP jacket can be calculated using the confined concrete strength. The maximum confined concrete compressive strength \( f_{cc}' \) can be calculated from ACI.2R_08.

\[
f_{cc}' = \psi_f 3.3 K_a f_l + f_c'
\]

where
- \( f_c' \) (the peak concrete strengths for unconfined)
- \( \psi_f \) (an additional reduction factor) = 0.95
- \( K_a \) (shape factor) = 1
- \( f_c' \) (the effective strain level in the FRP at failure) = \( K e_{fu} = 0.586 e_{fu} \)
- \( n \) (the number of piles)
- \( f_l \) (the maximum confinement pressure) = \( \frac{2E_f t_f e_f}{D} \)

According to the remaining strength of the concrete column after the fire damage mentioned above, the concrete compression strength could be calculated, as presented in Table 3.

<table>
<thead>
<tr>
<th>( f_c' ) (ksi)</th>
<th>( f_c'' ) (ksi)</th>
<th>reduction ( f_c' ) (ksi)</th>
<th>( f_{cc}' ) (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.00</td>
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<td>2.87</td>
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CONCLUSIONS

CFRP can be utilized as strengthening technique for increasing the axial load carrying capacity of fire damaged circular reinforced concrete columns. The axial compressive strength of fire damaged concrete increases significantly by warping with CFRP composites. The amount of strength gained from CFRP can be calculated according to the ACI.2R_08. The amount and warping method of CFRP for certain RC columns damaged by fire depends on the column type and condition. Further study and experiments are needed for assessing the remaining strength of the CFRP strengthened fire-damaged concrete structures.

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AXIAL STRENGTH OF FRP-CONFINED REINFORCED CONCRETE COLUMNS

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ABSTRACT

The enhancement of the axial strength of plain concrete columns wrapped with fibre-reinforced polymer (FRP) fabrics has been extensively investigated, both analytically and experimentally. Respective research on reinforced concrete (RC) columns strengthened with FRP is significantly more limited. This paper deals with axially loaded RC columns fully wrapped with carbon and glass FRP, with circular, square, and rectangular cross-section. A simple design model is proposed and described in detail, which has resulted from a combination of Eurocode provisions in such a way that the predictions are good and also safe. Furthermore, three models from the literature, selected among others because of their good agreement with experimental data, are also presented and some of their aspects are discussed. All models are applied to 95 specimens tested in the literature. The criteria for selecting the models presented here were their simplicity in application, combined with their accuracy in predicting the strength of test data available in the literature.

KEYWORDS

FRP, RC column, reinforcement, models, axial strength.

INTRODUCTION

Available design models that calculate the ultimate strength of reinforced concrete (RC) columns strengthened with fiber-reinforced polymer (FRP) fabrics are based on a variety of approaches. The models propose, in general, a procedure that takes into account the lateral confining contribution of both factors: steel stirrups and FRP. They differ, though, in the approach they adopt to calculate this contribution. The lateral confinement pressure because of steel confinement is usually taken into account by the Mander et al. (1988) model, by calculating the effectiveness in confinement according to the layout of stirrups in height and in the plane of the cross-section (CEN 2004a; Wang and Hsu 2007; Pellegrino and Modena 2010; Rousakis and Karabinis 2008; Galal et al. 2005; Illiki et al. 2008), or by a different procedure (Tastani et al. 2006), or not taken at all into account (Esfahani and Kianoush 2005). The lateral confinement pressure of FRP is calculated usually by the well-known formula yielding of stress equilibrium in the cross-section (Mirmiran and Shahawy, 1997), or by empirical equations (Rousakis and Karabinis 2008). A basic parameter that leads to different results is the value of rupture strain of the FRP used to calculate the maximum lateral pressure offered by the FRP (for which the value of 60% of the ultimate tensile strain determined by the direct tension tests was proposed by Lam and Teng, 2003). Furthermore, in case of rectangular cross-sections, different empirical factors are introduced in each model in order to account for the reduced confinement provided (Pellegrino and Modena 2010). Finally, the models differ considerably in the way they calculate the ultimate confined concrete strength from the lateral pressure. Attempts have been made also to take into account the negative effect of buckling of the longitudinal steel bars (Pellegrino and Modena 2010; Bai et al. 2012; Tastani et al. 2006) which may lead to premature failure. This paper presents four models, among a number of models considered elsewhere (Paparizos 2016; Miliokas 2016), that were found to lead to the best predictions of an experimental database of 95 specimens from the literature. In detail is presented only the model proposed, which has resulted from a combination of Eurocode provisions in such a way that the predictions are good and also safe. From the three other models presented only particular issues are discussed, while all the details may be found in the original publications.

PRESENTATION OF SOME ASPECTS OF THE MODELS CONSIDERED

Eurocodes do not explicitly describe how to estimate the ultimate axial capacity of FRP-confined RC columns, although EN1998-3 offers equations to separately calculate the confinement effect of steel and FRP. Furthermore, alternative procedures are offered to calculate the lateral confinement pressure due to FRP. Therefore, the model presented in this paper is the result of the combination of provisions in EN1998-3 (CEN 2005), EN1998-1 (CEN 2004a) and EN1992-1-1 (CEN 2004b). This calculation procedure has been found to lead to good and safe predictions (Figure 1a, Table 2) and will be presented in detail in the following. The stress-strain relation of confined concrete compressive strength, \( f_{ck} \), in relation to the effective lateral compressive strength, \( \sigma_2 \), and the characteristic compressive concrete strength of concrete at 28 days, \( f_{ck} \), is calculated according to Eqs 1a and 1b from EN1992-1-1 (CEN 2004b):

\[
\begin{align*}
    f_{ck,a} &= f_{ck}(1.0 + 5.0\sigma_2 / f_{ck}) & \text{for } \sigma_2 \leq 0.05 f_{ck} \\
    f_{ck,a} &= f_{ck}(1.125 + 2.5\sigma_2 / f_{ck}) & \text{for } \sigma_2 > 0.05 f_{ck}
\end{align*}
\]

The confinement pressure applied by the FRP sheet is calculated from Eq. 2:

\[
f_{f,frp} = 2E_f \varepsilon_{ju} / D
\]

where \( E_f \) is the FRP elastic modulus, \( \varepsilon_{ju} \) is the ultimate strain of FRP, \( t_f \) is the total thickness of the FRP jacket, and \( D \) is the diameter in case of a circular cross-section, or the larger cross-section width in case of rectangular cross-sections. It is noted that EN1998-3 (CEN 2005) specifies that \( \varepsilon_{ju} \) is “the adopted FRP jacket ultimate strain, which is lower than the ultimate strain of FRP \( \varepsilon_{ju} \) ”, setting elsewhere 0.015 the limit strain for CFRP. However, since no effectiveness factor is explicitly mentioned in the code, the model has been applied in this work using the ultimate strain for the FRP materials. To account for the increased axial strength this assumption would result, the contribution of longitudinal reinforcement is not included in calculating the ultimate axial strength. It is noted that the lateral strains measured on the FRP jacket in column tests, have shown considerable variation along the same cross-section (Pellegrino and Modena 2010 and Wang et al. 2012). Therefore, it is difficult to decide which value of the FRP rupture strain is more representative to adopt for the calculation of the confining lateral pressure at the ultimate load. The values of the in-situ jacket rupture strains adopted in the various models vary, e.g. Lam and Teng (2003) assume \( \varepsilon_{ju} = 0.586 \varepsilon_{ju} \), while Illiki et al. (2008) assumed \( \varepsilon_{ju} = 0.85 \varepsilon_{ju} \), both studies for carbon FRP jackets. Both models in these two papers show very good correlation with the experimental results.

The confinement pressure due to steel stirrups, \( f_{s,sl} \) is proposed to be calculated by Eq. 3 (among different alternatives supplied in EN1998-3 (CEN 2005):

\[
f_{s,sl} = a \rho_{sx} f_{yw}
\]

where \( f_{yw} \) the stirrup yield strength (in MPa), \( \rho_{sx} = A_{sx} / b_h s \) =ratio of transverse steel parallel to the direction x of loading, with \( s \) the stirrup spacing, and \( a \) being the confinement effectiveness factor, calculated according to Eqs 4a, 4b and 5, for circular and rectangular cross-sections, respectively (EN1998-1, CEN 2004a):

\[
\begin{align*}
    a &= (1 - s / 2d_h) & \text{circular cross-section with spiral hoops} \\
    a &= (1 - s / 2d_h)^2 & \text{circular cross-section with circular hoops} \\
    a &= (1 - 6h_i b_h / b_h) / (1 - s / 2d_h) & \text{rectangular cross-section}
\end{align*}
\]

where \( d_h \) is the diameter of the confined circular core (to the centreline of hoops), \( h_i \) and \( b_h \) are the depth and width, respectively, of the confined rectangular core (to the centreline of hoops), \( s \) is the spacing of hoops, and \( b_h \) is the distance between consecutive longitudinal bars, engaged by a tie or a stirrup.

The confined concrete strength, \( f_{ck,a} \), is calculated according to Eqs 1a and 1b, by calculating the total lateral confinement pressure due to both FRP (Eq. 2) and stirrups (Eq. 3):

\[
\sigma_2 = f_{f,frp} + f_{s,sl}
\]

Pellegrino and Modena model (2010)

The model is applicable to columns with and without steel, for all types of cross-sections. It has been calibrated against a vast number of specimens with and without steel reinforcement and takes explicitly into account a number of parameters that other models do not consider. The authors propose the use of an empirical formula to calculate the coefficient of efficiency, \( k_e \), of the FRP, i.e. to calculate the actual tensile FRP-rupture strain, \( \varepsilon_{ju} \) from the ultimate strain of FRP \( \varepsilon_{ju} \), \( \varepsilon_{ju} = k_e \varepsilon_{ju} \). For RC elements \( k_e \) considers the effect of FRP stiffness on restraining
steel buckling, is calculated according to Eq. 7, and depends on parameter \( C \) that expresses the ratio of mechanical steel percentage \( (E_{y,\text{long}} \cdot \rho_{y,\text{long}}) \) to the mechanical FRP percentage \( (E_f \cdot \rho_f) \), where \( E_{y,\text{long}} \) and \( \rho_{y,\text{long}} \) are the elastic modulus of the longitudinal steel reinforcement and the longitudinal steel ratio, respectively, \( E_f \) is the elastic modulus of FRP and \( \rho_f = 4t_f / D \) for circular- and \( \rho_f = 2t_f (b + h) / bh \) for rectangular cross-sections, with \( D, b, h \) the geometric dimensions of the cross-section.

\[
k_u = \gamma = \frac{E_{y,\text{long}} \cdot \rho_{y,\text{long}}}{E_f \rho_f} \leq 0.8
\]

where \( \gamma = 0.7 \) for CFRP and \( \gamma = 1.5 \) for GFRP fabric.

**Rousakis and Karabinis model (2008)**

The model is applicable to circular and square cross-sections. Formulas are offered (from regression analysis) to calculate the confined strength due to FRP. \( f_{cc,\text{FRP}} \) in which tensile FRP-rapture strain \( \varepsilon_{ju} \) is not needed: Eqs 8 and 9, for round and square cross-sections, respectively. The total confined strength of the element is calculated as sum of the independent contribution of FRP, of stirrups and of longitudinal reinforcement.

\[
f_{cc,\text{FRP}} / f_{co} = \left( \rho_f E_f / f_{co} \right) \left( 0.4142 E_f \cdot 10^{-6} / E_{yu} + 0.0248 \right) + 1
\]

\[
f_{cc,\text{FRP}} / f_{co} = 2 \left( \rho_f E_f / f_{co} \right) \left( 0.4142 E_f \cdot 10^{-6} / E_{yu} + 0.0248 \right) / (2r/b) + 1
\]

where \( \rho_f = 4t_f / D \), and \( D, b \) and \( r \) are the diameter, external dimension of square section, and the corner radius, respectively.

**Ilki et al. model (2008)**

Applicable to CFRP-confined RC elements, this model calculates the confined concrete strength as sum of the enhancement due to (a) CFRP (Eq. 10), (b) internal transverse reinforcement (ITR, Eq. 11) and (c) the unconfined concrete strength of the RC specimen, assumed to be 85% of the standard cylinder strength at the time of testing if the strength of the same unconfined specimen is not available.

\[
f_{cc,\text{FRP}} = f_{co} + 2.54 f_{I,\text{FRP}}
\]

\[
f_{cc,\text{ITR}} = f_{co} + 4.54 f_{I,\text{ITR}}
\]

where \( f_{I,\text{FRP}} \) the effective lateral confinement stress, calculated according to Wang and Restrepo (2001) for CFRP, and a \( f_{I,\text{ITR}} \) the lateral stress for ITR confinement, according to Mander et al (1998). Tensile rupture CFRP strain is assumed to be 85% of the ultimate strain \( \varepsilon_{fu} \).

**COMPARISON WITH EXPERIMENTAL RESULTS**

The data used to evaluate the predictive capacity of the models consist of 95 specimens, with circular, square, and rectangular cross-sections. The specimens considered had, in general, steel reinforcement and were wrapped with FRP fabric extending to the whole height of the column: thirteen specimens with glass FRP fabric (GFRP), and the rest with carbon FRP fabric (CFRP). The test data is taken from the following publications: Bai et al. (2012), Bourmas et al. (2007), Carey and Harries (2005), Chaalal and Shahawy (2000), Chastre and Silva (2010), Cole and Belbari (2001), De Luca et al. (2011), De Paula and Da Silva (2002), Esfahani and Kianoush (2005), Feng et al. (2002), Harajli et al. (2006), Ilki et al. (2008), Li et al. (2003), Matthys et al. (2006), Rodriguez and Silva (2001), Roussakis and Karabinis (2008), Tastani et al. (2006), Wang and Hsu (2008), and Wang et al. (2012). The range of the test parameters in the database included are grouped in Table 1. It is noted that the range of the test parameters provided is purely indicative, since only the precise values of all parameters used in each specimen may be representative of the actual behavior of the specimen.

In Figures 1a, 1b and 2a, 2b, the axial strength from the test (“experimental”) is compared to the respective value predicted by each design model considered (“model”). All values are expressed in MPa. In the model of Rousakis and Karabinis (2008) the specimens with rectangular cross-section are not included (22 specimens), and in the model of Ilki et al. (2010) the specimens with GFRP are not included (13 specimens), because of the limitation of applicability of the models. It is noted that the strengths of materials were entered in the models without reduction factors, since they were obtained experimentally. Values below the diagonal are unsafe predictions, given that the model estimates higher capacity than the one that was observed in the test. However, the application of safety
factors used for design would result in that certain unsafe predictions would be on the safe side. The main objective of the comparison, however, is to determine which model tends to better predict the ultimate axial strength for the range of the test parameters considered.

**Table 1** Range of test-parameters of the database

<table>
<thead>
<tr>
<th>Test parameter</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined concrete compressive strength (MPa)</td>
<td>15 to 46</td>
</tr>
<tr>
<td>Ratio of longitudinal reinforcement, ( \rho_s )</td>
<td>0.0017 to 0.014</td>
</tr>
<tr>
<td>Ratio of transverse reinforcement, ( \rho_w )</td>
<td>0.0026 to 0.0164</td>
</tr>
<tr>
<td>Ratio of FRP fabric, ( \rho_{vf} )</td>
<td>0.0022 to 0.0847</td>
</tr>
<tr>
<td>Number of CFRP -layers</td>
<td>1 to 6</td>
</tr>
<tr>
<td>Number of GFRP -layers</td>
<td>1 to 6</td>
</tr>
<tr>
<td>(Stirrup distance, ( s )) / (longitudinal bar diameter, ( D_s ))</td>
<td>3.6 to 47</td>
</tr>
<tr>
<td>Yield strength of transverse reinforcement (MPa)</td>
<td>200 to 587</td>
</tr>
<tr>
<td>Yield strength of longitudinal reinforcement (MPa)</td>
<td>275 to 620</td>
</tr>
<tr>
<td>Diameter of circular cross-section (mm)</td>
<td>150 to 610</td>
</tr>
<tr>
<td>Dimension of rectangular cross-section (mm)</td>
<td>79 to 610</td>
</tr>
<tr>
<td>Aspect ratio of sides of rectangular cross section</td>
<td>1 to 2.7</td>
</tr>
</tbody>
</table>

**Table 2** Predicted-to-test axial strength ratios of FRP-confined RC columns

<table>
<thead>
<tr>
<th>axial strength model</th>
<th>Average Predicted-to-test axial strength ratio</th>
<th>Coefficient of determination ( R^2 )</th>
<th>Correlation coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eurocodes (CEN 2004, 2005)</td>
<td>0.927</td>
<td>0.893</td>
<td>0.945</td>
</tr>
<tr>
<td>Pellegrino and Modena (2010)</td>
<td>0.940</td>
<td>0.841</td>
<td>0.917</td>
</tr>
<tr>
<td>Rousakis and Karabinis (2008)</td>
<td>0.974</td>
<td>0.781</td>
<td>0.884</td>
</tr>
<tr>
<td>Ilki et al. (2008)</td>
<td>0.960</td>
<td>0.842</td>
<td>0.918</td>
</tr>
</tbody>
</table>

(a) Model according to Eurocodes
(b) Model of Pellegrino and Modena (2010)

Figure 1 Axial strength predictions for the whole data base

Table 2 offers statistical indices for the models presented. All models predict well the experimental database. The model based on Eurocodes results in the best correlation coefficient and \( R^2 \)-value, but lower average predicted-to-test axial strength ratio among the models considered. In general the predictions of this model are on the safe side, hence the lower average predicted-to-test axial strength ratio. Given the fact, though, that it is a code-model, as such, it should provide increased safety. Therefore, the Eurocode model is recommended for use in design because it combines a uniform and simple approach for all types of cross-sections and FRP materials, and describes well the influence of the various design parameters.
CONCLUSIONS

The proposed model is based on Eurocode provisions and makes the simplistic assumptions that a) the FRP strain at failure is assumed to be the one provided by the manufacturer from flat coupon tests, and b) the longitudinal reinforcement is not disregarded in the calculation of the axial strength, in order to counterbalance for the overestimation of the axial strength because of the increased rupture strain of the FRP. The predictions of all the models presented have shown good agreement with the experimental results of the test database used in this study. However, the proposed model is recommended for use not only because of its slight superiority over the other models concerning its predictive capacity to accurately describe the behaviour of all design parameters, but mainly owing to its simplicity of application and uniform approach to all types of cross-sections and FRP materials.

REFERENCES


ABSTRACT

Fiber Reinforced Polymer (FRP) can be used as an excellent material for structural retrofitting to against seismic or impact loads and has been applied widely in the past 20 years to confine reinforced concrete column. Due to concentrated stress at the corners of square or rectangular columns which significantly influences FRP confinement, various modification methods which modifying the column cross sections have been investigated recently. This paper aims to provide an overview of the recent investigations on confining concrete columns with non-circular cross section using various modified cross section methods. The characteristic of FRP in confining existing columns in bridges and buildings was summarized first in the paper, followed by the assessment of performance of circular and non-circular columns confined by FRP. Investigations on modifying column cross section from square or rectangular to circular or elliptical sections before being wrapped with FRP layers using different techniques are reviewed in details. The review revealed that modifying the column cross section using grouting material with prefabricated FRP is an effective way to reach desired effectiveness of FRP confinement in strengthening existing columns.

KEYWORDS

Confinement, non-circular cross section column, FRP wrapping, modifying cross section, review.

INTRODUCTION

Many structures had built before the modern code of design, so it cannot withstand severe seismic activities. It is anticipated that, if no treatment for these buildings is taken, a one third of them will be threatened to fail over the next four decades due to seismic loading (Promis & Ferrier 2012). The demand to strengthen seismic resistance of concrete columns can be achieved in one of three ways: improve strength, improve ductility, or improve both ductility and strength. However, the type of structures is the criterion for choosing the strengthening method. Often, the method of improving ductility is considered as the favorite method to dissipate the energy of seismic activity (Promis & Ferrier 2012). Confining concrete structure by high strength materials externally can enhance many properties significantly such as ductility, energy dissipation as well as concrete strength. To date, the performance of many structural elements has been enhanced at different aspects by using FRP for its superior strength and high resistance against corrosion (Teng & Lam 2004; Gu et al 2010).

Confining concrete elements might include wrapping the existing concrete by FRP as a retrofitting layer or encapsulating a new pre-fabricated system (Mirmiran & Shahawy 1997). Using FRP to rehabilitate columns has many advantages, however these advantages could be less valuable and effective when it was used to repair a square or rectangular column. The tests that have been reported in the literature have established that the corner radius significantly affects the confinement effectiveness (Wang and Wu 2008). FRP external sheet can enhance the ductility and strength of concrete columns with circular cross section due to the uniform distribution of pressure that is exerted surrounding the column. Whereas, FRP confining layer cannot provide a notable confinement on the straight face on rectangular or square column, the effectiveness depends on the curvature of the corners. FRP wrapping can provide significant advantages only when column cross section shape is changed to elliptical or circular shape. In the last decades, a large amount of investigations have been conducted on circular columns confined by FRP. However, the investigations on FRP confined square or rectangular columns were much less. In square column, the confinement effectiveness is less than that of circular column due to the non-uniform distribution of confinement to concrete. Some investigations reported that there was no confinement at sharp corners, others researchers mentioned that a certain degree of confinement was provided by a jacket with sharp corners (Mirmiran 1998; Wang & Wu 2008).
A study presented by Mirmiran (1998) revealed that cross section of column had direct effects on the confinement performance. As external FRP layers had to be bonded on a column surface, they must be bent. The bending affects the efficiency of FRP laminates, the effectiveness of confinement of column depends on the corner curvature (Yang et al. 2001). Confining effectiveness for circular column is more than their square counter parts due to the concentration stress at the corner (Mirmiran 1998; Mostofinejad & Ilia 2014). Al-Salloum (2007) revealed that FRP confinement efficiency and performance were absolutely related to the curvature radius of column edges which may cause premature failure of FRP rapture at the sharp edges. He also found that the failure of square column started at the sharp edges that the stress was concentrated. On this basis, investigators have studied the effect of column cross section on the efficiency of the FRP confinement. This study presents a review of the effectiveness of various cross section modification methods when the columns were strengthened with FRP wrapping or jacketing.

THE EFFECT OF EDGE CURVATURE ON COLUMN CONFINEMENT

An experimental investigation was presented by Al-Salloum (2007) on the effects of curvature on the strength of confined square columns. The investigation included testing columns with a wide range of corner radius in addition to circular column to compare the performance. The results revealed that the circular specimens presented the best efficiency. The poorest performance was the column with 5mm edges radius. As the cross section of column changed from circular to square shape, the ultimate strength decreased because of the non-uniform stress distribution. The effect of corner radius on CFRP confinement efficiency was also investigated by Wang & Wu (2008). The effect of CFRP thickness, concrete strength and corner radius was studied specially for polygonal columns as shown in Figure 1. The results indicated that corner radius is of great importance in relation to the level of confinement. The strength gain of confined concrete columns was in direct linear relationship to the corner radius ratio regardless of the confinement level and concrete strength. The results also revealed that, the confinement effectiveness was insufficient to enhance the columns strength before the peak load. On the other hand, a higher level of confinement does not correlate with the higher ductility; in contrast, it reduced the ductility of the column in some cases.

CONFINEMENT TECHNIQUES FOR COLUMNS WITH NON-CIRCULAR CROSS-SECTION

It was clear that the corner radius had a significant effect of the effectiveness of column confinement, different types of techniques were then developed by various researchers to modify the cross section of column before wrapping with FRP to avoid stress concentration on sharp edges. The most effective techniques are discussed and compared in the following sections:

Rounding the Sharp Edges of Columns

Rounding the sharp edges of columns has been considered as a conventional technique to enhance the performance of FRP confinement in the last decade (He et al. 2013; Yaqub and Bailey 2012; Wu & Wei 2010). This technique can help to reduce the punching and concentration of FRP sheet at the sharp corners. Yaqub and Bailey (2012) investigated confining square concrete column by a single layer of carbon or glass FRP under seismic load after rounding the corners to be an arc with the radius of 25mm as shown in Figure 2. An experimental investigation comprising large scale of five damaged columns with various damage patterns was conducted by He et al. (2013) to assess the effectiveness of using carbon FRP for confining these columns. To prepare the damaged columns for CFRP installation, the loose concrete layer was removed and substituted by accelerated pastes. The sharp corners of the columns were rounded with the radius of about 25 mm by using a hand grinder. Yaqub and Bailey (2012) and Teng and Lam (2002) mentioned that the effectiveness of FRP confinement gradually increased with an increment in the radius of edges. Choosing 25 mm radius is more suitable with using hand grinder as recommended by these researchers. With increasing the edges radius, the stress distribution becomes more uniform, and this would improve the column strength and ductility. However, as the radius was limited by the presence of internal steel reinforcement to a small value, the confinement effect was also limited (Teng and Lam, 2002). The efficiency
of FRP confinement of square columns with round corners is still significantly lower than circular columns (Al-Salloum, 2007).

Figure 2 Rounding edges (Yaqub and Bailey 2012)  
Figure 3 Circularization and CFRP (Pham et al. 2013)

FRP Wrapping and Shape Modification

The idea of column shape modification was explored by Priestley et al. (1994) for seismic retrofit of bridge columns using steel jackets, with the gap between the column and the steel jacket filled with concrete (Teng and Lam 2002). More research has been carried in recent years of using FRP (Alsayed et al. 2014; Hadi et al. 2013 and Pham et al. 2013) where, rectangular/square column was modified to an elliptical or circular cross section by adding concrete or mortar before wrapping FRP confinement.

Shape modification with circularization

Circularization technique has been commonly used for shape modification of column with a square cross section. Pham et al. (2013) used concrete strength ranging from 40 MPa to 100 MPa to change a square cross section of 150x150 mm to a circular shape before the column was wrapped by CFRP as shown in Figure 3. The concrete cover and CFRP layer behaved as a composite material under concentric and eccentric compressive load and flexural load. Normal concrete with design compressive strength of 40 MPa was used to cast the specimens (core). Unidirectional CFRP, with the width of 75mm, was used for confinement. In order to create a circular segment with the chord of 150 mm (the same length as square column side), sections made of cork were used and added to the rectangular mould to create the required shape. The segments, after 28 days of casting, were removed and bonded to the columns using adhesive after cleaning the columns and the segments. Four circular covers were bonded onto the column side to change the cross section from square to circular. The specimens were then confined in three layers of separate rings of CFRP with the overlap of 100mm. Different types of load, such as concentric, eccentric, and flexural load were considered to study the behavior of modified square columns.

The results revealed that, the un-strengthened columns failed by dedonding of concrete cover and buckling of the main reinforcement at the mid height of the column for concentric load. In contrast, brittle failure was observed with crushing of concrete covers and rupture of CFRP for confined columns (Figure 4). For eccentric force, cracks were observed on the tensile sides between two strips placed at the mid height of the column. The specimens also failed by the rupture of CFRP followed by compression failure in concrete. For flexural load, vertical cracks were observed in the mid span of specimens. The results also revealed that confined specimens presented high ductility with respect to reference group. The strength of the groups which strengthened in segment with 80 and 100 MPa concrete presented similar behavior as that of the columns strengthened in 40 MPa. The research indicated that the effect of segments compressive strength on the ductility of confined columns is negligible. The same technique was also used by Hadi et al. (2013). It was recommended that this method proved to be effective in increasing the ultimate load for the confined column.

The effectiveness of shape circularization was investigated by Hadi et al. (2013), where three techniques to confine square section column were compared. The first method (RF) was that the column’s corners were rounded to have 20 mm radius before wrapping the columns in three layers of carbon fiber. For comparison, the straight faces of the columns in the second method (CF) were combined with segments of circular section to change the shape of column to circular cross section before wrapping with three layers of CFRP. In the third one (CS) the same technique as that of the second method was considered but steel strips were used to warp the columns. The columns were left without confinement as reference columns (N). The testing results are shown in Figure 5.
The testing results indicated that Group CF (using shape circularization and CFRP wrapping) outperformed the other groups because of the significant reduction of stress concentration at the corners of square columns and using CFRP. Specimens of group CF showed the highest ultimate load capacity for all types of loading conditions followed by specimens of group CS (using shape circularization and steel strips). Under eccentric load condition, the specimens of group CF also showed the highest strength as compared to other groups and same result for flexural test. It was proved that the concept of shape circularization to be an effective process to increase the ultimate load capacity of the columns confined by CFRP. The efficiency of the circularization process was higher than that of rounding corners process.

**Shape modification with an elliptical section**

For columns with rectangular shape, modifying the column section into an elliptical section has been practiced. The compressive strength and stress–strain behavior of FRP-confined concrete in elliptical columns have been investigated (Teng and Lam 2002). The testing results indicated that for CFRP-wrapped specimens, the compressive strength was significantly affected by the major-to-minor axis length ratio a/b of the column section (Figure 6). The confining FRP becomes increasingly less effective as the section becomes more elliptical. However, substantial strength gains from FRP confinement can still be achieved even for strongly elliptical sections.

Large scale axial compression tests were performed on RC columns with and without CFRP confinement (Alsayed et al. 2014). The CFRP was wrapped on the column after modifying the rectangular section to an elliptical shape using normal cement mortar. Failure of strengthened column was due to the crushing of concrete and buckling of longitudinal rebars after yielding instead of the rupture of CFRP sheet which was reported in many studies. The bulging of column at mid-height and a simultaneous rapid decrease in the load carrying capacity of strengthened column was an indication that the actual failure was due to crushing of concrete and buckling of longitudinal steel (Alsayed et al. 2014).

Another effective technique to help reducing the stress concentration at column corners and increase the efficiency of FRP confinement has been investigated (Mahfouz et al. 2001). A "Sandwich Wrapping Confining System (SWCS)" for strengthening columns having a rectangular cross section was developed where two layers of FRP with incompressible pulp core were used to confine rectangular concrete columns using the normal epoxy wrapping method. It was found that the stiffness of the SWCS in the transverse direction of the element must be high enough to bring about the required confining pressures at relatively low transverse strains. SWCS has both axial and flexural stiffness in the transverse direction of the column, it effectively improved the properties of square and rectangular columns.
Shape Modification with Prefabricated FRP Composite Systems

Prefabricated FRP composite repair systems may be used both as the formwork for concrete casting to achieve shape modification and to provide the lateral confinement following the curing of the new filling materials (Teng and Lam 2002). Yan and Pantelides (2011) attempted using prefabricated FRP shell and expansive cement concrete to retrofit non-circular concrete columns. As expansion of the grout was restrained by FRP shell, the post-tensioning effect would generate an active confinement. The testing results indicated that columns strengthened by the non-bonded FRP jacket with expansive cement achieved a higher axial strength compared to columns strengthened with FRP wrapping method. The failure mode was due to facture of the FRP shell and cracking of the expansive cement (Figure 7) which indicated extensive participation of the FRP in confinement.

Herwig & Motavalli (2012) used unbonded GFRP and light weight concrete to confine square columns. The sides of the columns were mounted by light weight deviation concrete board before the GFRP confinement as shown in Figure 8. The results revealed that for the columns confined in 6 and 7 plies, the ultimate strength was about 46% and 32% higher than the un-confined column. It was found that the effectiveness of the confinement was determined by the accurate choice of board, unbonded spies and the wraps geometry and number. The method could be used for retrofitting existing columns as an economic technique to reach a desired ductility and high resistance against impact loading.

CONCLUSIONS

Enhancing the strength and ductility of columns are one of the important applications for FRP, which can be used as a prefabricated jacket or external wrapping on the column. The advantages of FRP reinforcement for concrete column have been studied through many investigations. FRP wrapping can effectively enhance the ductility and strength of concrete columns with circular cross section due to uniform distribution of the pressure. Whereas, FRP cannot provide a notable confinement on the straight face on rectangular or square columns. For such a case, FRP sheet can only provide significant advantages when the column cross section shape was modified to elliptical or circular shape. The research results indicated that the method of modifying column cross section before FRP wrap was more efficient than rounding the sharp edges of columns. Another method which uses prefabricated FRP composite as the formwork for concrete casting to achieve shape modification and to provide the lateral confinement following the curing of the new filling materials has been developed recently. Using prefabricated FRP with grouting materials could be considered as an effective way in enhancing column strength in existing buildings, however, the research in this area has been limited. It is important to focus on research in this area in the future to find out a suitable grouting material that can be used as part for the new FRP composite system.

REFERENCES


AXIAL COMPRESSIVE BEHAVIOR OF LARGE-SCALE SQUARE COLUMNS CONFINED WITH WRAPPED CFRP

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ABSTRACT

Based on a series of tests on seven groups of CFRP confined large-scale square concrete columns with different corner radius, the basic mechanical properties were obtained. Test results demonstrate that the confined strength increases according to the corner radius increasing except for the specimens with sharp corner. All of the specimens failed due to CFRP rupture and the fracture location occurred in the chamfer region. So the phenomenon of the stress concentration can be observed obviously. The ultimate strains measured on the CFRP jackets are about 35% less than the ultimate strain tested by the coupon specimens, which is more serious than the smaller-scale specimens. Five existing confinement models are used to predict the ultimate compressive strength of test columns in the paper and the results present significant differences.

KEYWORDS

CFRP, confinement, square column, axial compression.

INTRODUCTION

Wrapping fiber reinforced polymer (FRP) is an effective method to improve the mechanical properties for concrete columns. The tensile forces of FRP jacket around the perimeter of the columns confine the lateral expansion of concrete and thereby enhance their deformability and axial capacity significantly. Some experimental research have been performed on FRP confined square or rectangular columns in the recent years (e.g., Mirmiran et al. 1998; Pessiki et al. 2001; Shehata et al. 2002; Rocca 2007; Wang and Wu 2008; Wang et al. 2012). Most of reported experiments has focused on the small-scale specimens, especially with the length less than 250 mm and height less than 600 mm. The research conducted by Pessiki et al. (2001) and Masia et al. (2004) has shown that the enhancement effect by CFRP confinement in strength and ductility observed for the small-scale square specimens should not be directly extrapolated to the real size columns. In order to address the knowledge gaps discussed above, this paper carried out a series of tests on CFRP confined large-scale square section columns. This study try to assess the influence of column cross-sectional size on the strength and deformation of CFRP confined square columns.

EXPERIMENTAL DESIGN

Specimen Design and Material Characteristics

The experimental program consisted of testing of 42 large-scale plain concrete columns under concentric axial load, three specimens in each group. Half of columns were confined by wrapped CFRP jackets and the rest served as control (unwrapped). All columns are square section with 300mm in side length b and 600mm in height. The specimens vary with a corner radius r of 0, 15, 30, 60, 90, 120 and 150mm according to the corner radius ratio rc, corner radius ratio and equal to 2r/b, of 0, 0.1, 0.2, 0.4, 0.6, 0.8 and 1.0. It means that the columns with square (sharp) section or circular section with rc equal to 0 or 1.0, respectively. From the rc point of view, this study can be good corresponded to the series tests on the small-scale specimens performed by Wang and Wu (2008), of which the cross section sizes are 150mm in side length and 300mm in height.

The specimens were prepared using a single mix design and cast in the laboratory, in which mix proportion of materials was selected as cement : water : sand : gravel = 1:0.56:1.90:3.10. The 28-day average compressive cylinder strength was 38.4MPa according to the ACI standard (2008a).
An unidirectional carbon fabric, with the nominal thickness of 0.167mm, and a two-part epoxy resin were selected for wrapping concrete columns. To determine the material properties of the CFRP, six flat coupons of 25mm width and 200mm length were test in accordance with ASTM D3039 to produce the tensile strength of 3587MPa, the elastic modulus of 236GPa and the elongation at rupture of 1.52%.

**Specimen Preparation**

A suit of high precision steel molds were custom-made, three identical molds for each corner radius. The concrete was poured into steel molds in three equal layers and vibrated by a vibration rod. Two days after casting, the specimens were removed from the formworks and cured in laboratory for 30 days.

The CFRP was wrapped in the hoop direction in a wet lay-up manner. Discontinuous three sheets were used to wrap a specimen, in which had a single lap 100mm in length for each layer. Those overlap regions were arranged near the middle of different sides in order to avoid the corners. The wrapped columns were stored in an indoor condition at least 4 weeks and tested at least 2 months after pouring of the concrete.

**Instrumentation and Test Procedure**

All tests were conducted using a YAW-10000 universal testing machine of 10000kN capacity, as shown in Figure 1. Axial deformation of specimens were measured with two linear variable displacement transducers (LVDTs) which were mounted onto the two steel frames that were spaced about half of the column height at the midheight region, as show in Figure 2. A certain amount of strain gauges of 10mm gauge length were bonded onto the outside surface of the wrap in the hoop direction at midheight section of each column.

All specimens were tested according to an automatic two-stage control mode. Columns were tested in a load controlled manner at a loading rate of about 5kN/s before 90% of the pre-estimated peak load reached. Then a displacement control was used at approximately 0.05mm/min until specimen’s failure.

**TEST RESULTS AND ANALYSIS**

**Failure Modes**

The typical failure modes of the test specimens are shown in Figure 3. The unconfined columns failed in a classic shear manner, a mainly diagonal crack crossing the specimen, as shown in Figure 3(a). All confined columns failed by tensile rupture of the CFRP jacket within the midheight region. Continuous cracking sound could be heard approaching failure followed by an explosive sound suddenly. CFRP rupture region varied from 190mm to 450mm along the height direction. For columns with a small corner radius \( r \leq 30 \text{mm} \), the breaking points always occurred near the center of the corner and the vertical profile of the cracks were virtually straight, as shown Figure 3(b)-3(d). For columns with a lager corner radius \( r \geq 60 \text{mm} \), the breaking points occurred near the center of the corner or the curvature changing point and the ruptures were rather jagged, as shown in Figure 3(e)-3(h). The exactly rupture locations of all columns except for circular section \( r = 150 \text{mm} \) were situated in the chamfered region. It suggested that the stress concentration caused by the corner have important effect on the failure of the CFRP jacket in the large-scale columns.
Stress-strain Behavior

The typical stress-strain curves for each group is presented in Figure 4, in which axial strain and lateral strain are defined as positive and negative, respectively. The compressive strength of all specimens are presented in Table 1.

![Unwrapped](image1)

![r=0mm](image2)

![r=15mm](image3)

![r=30mm](image4)

![r=60mm](image5)

![r=90mm](image6)

![r=120mm](image7)

![r=150mm](image8)

Figure 3 Failure modes of CFRP-confined concrete columns

<table>
<thead>
<tr>
<th>$r$ (mm)</th>
<th>$f_{co}'$ (MPa)</th>
<th>$f_{cc}'$ (MPa)</th>
<th>$f_{cc}'/f_{co}'$</th>
<th>CFRP strain (%)</th>
<th>$R_{max}$ (%)</th>
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Table 1 Compressive strengths of test

Test results from Table 2 and Figure 4 reveal that the confinement effectiveness of CFRP jackets are affected by corner radius $r$. The stress-strain curves of the specimens with small corner radius ($r \leq 30$mm) demonstrated postpeak softening behavior, whereas monotonically bilinear ascending responses presented for larger corner radius.
radius ($r \geq 60\text{mm}$). A turning point of the stress-strain curves for all confined columns can be found in Figure 4, which approaches the onset of the unconfined concrete strength. Similar stress-strain behaviors have also been presented in smaller square section columns (Wang and Wu, 2008).

Corner Radius Effect

Table 1 gives the result of enhancement ratio, $f_{cc}'/f_{co}'$, in which $f_{cc}'$ is the mean compressive strength of the CFRP confined columns and $f_{co}'$ is the compressive strength of the unconfined columns. The values of $f_{cc}'/f_{co}'$ increase from 1.05 to 2.20 as the section of confined columns changes from sharp corner to circle gradually. The five curves of $f_{cc}'/f_{co}'$ versus corner radius ratio $r_c$ are presented in Figure 5, included test results conducted by authors and Wang and Wu (2008). It can be seen that $f_{cc}'/f_{co}'$ raises with $r_c$ increasing and $f_{cc}'/f_{co}'$ is not showed significant improvement with sharp corner for all series tests. Compared with the curves of Wang and Wu (2008), the curve of this paper also discovery that the strength gain of the confined large-scale columns is almost proportional to the corner radius ratio even $r_c$ less than 0.2.

Rupture Strain of CFRP

The ultimate strain of CFRP jackets at failure are also listed in table 1. The strain values are average value of test data of the three identical specimens. Among them, $R_{\text{max}}$ is the ratio of the rupture values measured in large-scale columns and measured in flat coupons. From Table 1, the maximum of $R_{\text{max}}$ is 0.75. From all the statistical results of the specimens, the mean of rupture strains of CFRP jacket is less than 35% of the ultimate strain tested by material experiment. Compared with the data of Wang and Wu (2008), the maximum of $R_{\text{max}}$ in the group data is 0.88. and the mean of rupture strains of specimens decreases by 25% than the ultimate strain form the flat coupon test. It suggests that the confinement effect of FRP jacket deceases with the size of columns increase.

**COMPARISON OF THE TEST RESULTS WITH THEORETICAL MODEL**
Five existing confinement models, such as Mirmiran (1998), Lam and Teng (2003), Ilki (2004), Al-Salloum (2007) and Wu (2009), are used to predict the ultimate compressive strength of test columns in the paper. The comparative results between the theoretical predictions and tests are showed in Figure 6. According to the works of Ilki et al (2004), the hoop rupture strain of CFRP jacket was only taken as 70% of its ultimate tensile strain from the coupon test in the calculation models.

![Figure 6 Comparison between test results and existing models](image)

In the case of the small corner radius \( r_c \leq 0.2 \), the results predicted by Mirmiran (1998) and Wu’s (2009) strength model are good agreement with the test data. And other models are obviously overestimated the compressive strength of confined column. Under the condition of the larger corner radius \( r_c > 0.2 \), all models underestimate the compressive strength of the specimens and the prediction error increases significantly with the increase of \( r_c \).

CONCLUSIONS

1. Enhancement ratio \( f_{c'} / f_{c0} \) of the confined columns increases almost linearly with the increase of the corner radius ratio \( r_c \). Corner radius \( r \) has important influence on the mechanical properties of CFRP confined large-scale square column.
2. The exactly rupture locations of confined specimens are situated in the chamfered region. The measured ultimate strain of CFRP jackets is less than 35% of the ultimate strain tested by flat coupons. Compared with the test results from small size specimens, the confinement effect of FRP jacket deceases with the size of columns increase.
3. Five existing strength models of CFRP confined square column are used to predict the ultimate compressive strength of test columns. All models underestimate the compressive strength of the specimens and the prediction error increases significantly when corner radius ratio \( r_c \) is larger than 0.2.

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ABSTRACT

This paper presents an experimental study on the effect of coarse aggregate size on the mechanical behavior of passively confined concrete. In total, 32 concrete cylinders with different aggregate sizes and thicknesses of fiber reinforced polymer (FRP) jacketing were tested under monotonic axial loading. From the test results, it is shown that the aggregate size does not affect behavior of unconfined concrete and the beginning stage of stress-strain curves of FRP confined concrete. However, in the transitional stage of stress-strain relationship of FRP-confined specimens, the concrete with larger aggregates has a higher stress. The second stiffness of stress-strain relationship is not affected by the aggregate size. Furthermore, at the ultimate stage, larger aggregates cut FRP to failure more easily and earlier. Hence, the ultimate strain has a decreasing trend with increasing aggregate size. Meanwhile, the ultimate strengths are similar for the specimens with different aggregate sizes because of the two aspects with increasing aggregate size: enhancing effect on stress in the transition stage and decreasing effect on strain in the ultimate stage.

KEYWORDS

FRP, concrete, confinement, aggregate size, stress-strain relationship.

INTRODUCTION

The strength and ductility of concrete can be significantly increased by confining with fiber reinforced polymer (FRP). Both the strength model and stress-strain behavior are widely studied with large scatters in the recent decades (Ozbakkaloglu et al. 2013; Cao et al. 2016). The enhancing effect on the concrete mechanism behavior is caused by the FRP lateral confinement activated by the dilation of concrete material. That is to say, the mechanical behavior of concrete material under triaxial stress condition plays a key role in the FRP confining. For the concrete material, the coarse aggregate takes more than 75% of volume of concrete, and can affect mechanical properties of concrete (Neville 1995). To individually study the effect of aggregate size on the behavior of FRP-confined concrete columns, this work studied specimens by different sizes of coarse aggregate with fixing both the specimen geometrical proportions and their scales.

EXPERIMENTAL PROGRAM

Test specimens consisted of 32 circular concrete cylinders in total. The variation factors included mixture proportion, aggregate size, and number of FRP sheet layer. Details of the specimens are given in Tables 1. All the specimens were tested under uniaxial compression at a fixed load rate with monotonic displacement control.

Design of specimens

The size of all the cylinder specimens were 150 mm in diameter and 300 mm in height, which is widely used in stress-strain modeling of FRP-confined concrete (Lam and Teng 2002; Wu and Wei 2010; Wu and Jiang 2013a). In order to investigate the factor of aggregate size independently, and to avoid the “wall effect” (Neville 1995) caused by concentration of smaller aggregates near the surface of cylinders, coarse aggregate with only one size was used in a particular specimen.. Aggregate size was the major variable of the tests and values of 5, 10 and 20
mm were adopted. Though 20 mm aggregate in the cylinder specimens may disproportionately large for the practical large columns, the 20 mm specimens were studied in this work for the academic purpose. Carbon FRP (CFRP) was used with the variation of layer number from one to three plies. Two different mix proportions of concrete for 10 mm aggregate size especially were used which aimed to 25 MPa and 40 MPa for concrete uniaxial compressive strength. Two specimens were tested for each design. The test results in the following sections showed a very small difference between the two identical specimens in the two different groups.

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<th>$\varepsilon_{cc}$ ($\times 10^{4}$)</th>
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In Table 1, the two mix proportions are identified by the first letter A and B in the specimen ID; the second character of the specimen ID (d5, d10 or d20) denotes the coarse aggregate size of 5, 10 and 20 mm, respectively; and the third character of the specimen ID (p0, p1, p2 or p3) means the number of CFRP layers, where 0-ply is the unconfined concrete; the two identical specimens are separated into two different groups identified by the last number 1 or 2 in the specimen ID. For example, A-d10-p2-2 indicates the second identical specimen by mix proportion A, with 2-ply CFRP and 10 mm coarse aggregate.

A high tensile strength carbon fiber fabric known as TORAYCA Cloth, which has a nominal thickness of 0.167 mm, was used for jacketing. A two-part Sikadur-300 was used as saturant resin. Flat coupon tensile tests were
conducted to determine the material properties of the CFRP sheets. Mechanical properties of the CFRP laminate obtained from the coupon tests are 3922 MPa, 245 GPa and 1.6% in ultimate strength, tensile modulus and ultimate strain, respectively.

**Specimen preparation**

Mix A and B aimed to 25 MPa and 40 MPa respectively for the designed cylinder strength at 28 days. River sand was used as the fine aggregate. Crushed granite stones as coarse aggregate, which is a main parameter in this study, were sieved very carefully to divide three different sizes of aggregate, as shown in Figure 1(a). Coarse aggregates with a maximum size of 20 mm were sieved using a sieve level with \( 20 \times 20 \) mm\(^2\) square holes. The remaining aggregates on the 20 mm sieve level were used to cast the concrete with an aggregate size of 20 mm. Similarly, coarse aggregate with a maximum size of 10 mm and 10 mm sieve level were used to cast the concrete with an aggregate size of 10 mm. Then, the remaining stones were sieved continually to let them go through the sieve level of 6.3 mm and 5 mm, respectively. The stones left between such two levels, which indicated that their aggregate size was between 5 to 6.3 mm, were used to cast the concrete with an aggregate size of 5 mm. Therefore the exact average aggregate size should be \( (5+6.3)/2 = 5.65 \) mm. Both the fine and coarse aggregates were completely dried in an oven at 110°C for 24 hours to minimize the variation of water content. The concrete was mixed and cast in the laboratory. One day after casting, the specimens were removed from the molds and cured in a water tank at a constant temperature of 27°C for 28 days. Meanwhile, six cubes of \( 150 \times 150 \times 150 \) mm\(^3\) were prepared for each group and cured in the same manner as the cylinder specimens to monitor the development of concrete strength at different ages. At 28 days, the specimens were removed from the water and then ground with a grinding machine for getting a smooth contact surface.

The surface of specimens was cleaned with water and left to dry before FRP jacketing. The CFRP was wrapped around the specimens using the manual lay-up procedure, with the fibers oriented in the hoop direction. Each layer of CFRP has a single overlap of 150 mm in length. Details of the wrapping process can be found in Wu and Jiang (2013a). The wrapped specimens were kept in the laboratory air-conditioned environment at a temperature of 16°C for two weeks before testing.

![Aggregate sieving](image1)

![Test setup](image2)

**Figure 1** Compression test of specimens
Testing and instrumentation

The compression tests on FRP-confined concrete specimens were conducted using a 1600 kN MTS rock machine and a 2500 kN FORNEY testing machine, according to the load capacity of each specimen. Before testing, the calibrations of the load readings were conducted on the two testing machines. Two thick and flat steel plates were set at the top and bottom of the specimen and two linear variable differential transformers (LVDTs) were mounted on the left and right sides to obtain the axial deformation of specimen. Details are shown in Figure 1(b).

Digital image correlation (DIC) system that can capture continuous displacement and strain fields of the specimen surface during testing was used for strain measurement. DIC is very suitable for concrete testing and can provide much more accurate and reliable strain data than conventional electric strain gauges (Jiang et al. 2014; Wu and Jiang 2013b; Wu et al. 2015; Yu and Wu 2016). The instrumentation system includes two cameras and a computer used for storing and processing the captured image data. Two cameras were set in the front and behind the specimen in order to use DIC-2D system to measure the in-plane axial strain and displacement fields for the both sides. All the specimens were tested under a displacement control mode at a fixed rate of 0.3 mm/min.

FAILURE MODE

In the case of unconfined cylinder specimens, longitudinally splitting and shearing modes occurred randomly (Figure 2). The failure mode of FRP-confined columns was concrete crushing simultaneously after the CFRP rupture; no delamination of FRP at the overlapping zone was observed. Failure occurred suddenly at around mid-height, with an explosive sound. It can be seen that the aggregate size does not affect the failure mode for both unconfined and FRP-confined concrete columns.

EFFECT OF AGGREGATE SIZE ON THE BEHAVIOR OF UNCONFINED CONCRETE

In this study, for concrete with different aggregate sizes, the concrete mix proportion was fixed (mix A) to make the investigated factor more independent. Two different concrete mix proportions (mixes A and B) were adopted for one aggregate size (10 mm). The whole stress-strain curves for unconfined concrete were achieved in the tests as shown in Figure 3. In Figure 3(a), it can be observed that the elastic modulus (E) and strength (fcu) of concrete with the same mix proportion are quite similar. After the peak strength, the stress-strain curves of concrete with

![Figure 2 Failure mode of unconfined and confined concrete specimens](image)

![Figure 3 Strain-strain relationship of unconfined concrete](image)
smaller aggregates drop slower than that with larger aggregates. In Figure 3(b), the concrete with less water/cement ratio (mix B) shows a higher elastic modulus and strength, which is quite normal and common in concrete material.

EFFECT OF AGGREGATE SIZE ON THE BEHAVIOR OF FRP-CONFINED CONCRETE

The peak strength \((f_{cc})\) of both unconfined and FRP confined specimens, as well as the corresponding axial strain \((\varepsilon_{cc})\) are listed in Table 1. The concrete cylinder strength \((f_{co})\) and corresponding axial strain \((\varepsilon_{co})\) are obtained from the cylinder test data of the unconfined specimens. Typical stress-strain curves for all the specimens are plotted in Figure 4, where axial strain is achieved by DIC analysis.

\[\text{Figure 4 Effect of aggregate size on stress-strain curves}\]

The effect of aggregate size on the stress-strain relationship of FRP-confined concrete is investigated in this work. The stress-strain curves of confined concrete in this experimental program are drawn in Figure 4(a-c), with the individual FRP wrapping layers. Figure 4(d) enlarges the beginning part of Figure 4(a) to make the trend clearer. Subjected to adequate lateral confinement, all of the confined specimens have strain-hardening behavior.

At the beginning stage, the stress-strain relationships of specimens with various sizes of aggregates are the same. That is to say, the aggregate size does not change the initial stress-strain relationship. After the initial elastic stage, the concrete with larger aggregates has a higher stress from Figure 4. Furthermore, the second stiffness of stress strain curves is very similar for different kinds of concrete. However, the ultimate strain of confined concrete by larger aggregate size is smaller. This phenomenon is reasonable because the larger aggregates can cut the FRP more easily and earlier. Similar reasons and conclusions of decreasing on ultimate strain were conducted for higher strength concrete (Lim and Ozbakkaloglu 2013). Therefore, the ultimate strengths of confined concretes with different aggregate sizes are approximate according to the two opposite effect with increasing aggregate size: higher stress in transitional stress and lower ultimate strain.

CONCLUSIONS

An experimental investigation has been undertaken in this work to study the effect of aggregate size on the mechanical behavior of FRP confined concrete columns. The experimental program included loading tests of 32 column cylinders. After the analytical study, it can be concluded that the size effect by aggregate influences the mechanical behavior of FRP-confined concrete as:

1 For the unconfined concrete and the initial elastic stage of FRP-confined concrete, the aggregate size does not affect the mechanical behavior.
At and after the transitional stage of stress-strain relationship of FRP-confined concrete, the concrete by larger aggregates has a higher stress. The aggregate size has no effect on the second stiffness of stress-strain relationship.

In the ultimate stage, larger aggregates can cut the FRP to failure more easily and earlier. Hence, the ultimate strain has a decreasing trend with increasing aggregate size. The ultimate stresses are similar because of the two aspects: enhancing effect with increasing aggregate size in the transitional stage and decreasing effect with increasing aggregate size in the ultimate stage.

ACKNOWLEDGMENTS

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REFERENCE


STRESS-STRAIN BEHAVIOUR OF FRP-CONFINED CONCRETE IN ECCENTRICALLY-LOADED CIRCULAR COLUMNS

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ABSTRACT

Fibre-reinforced polymer (FRP) jacketing/wrapping has become a widely accepted technique for strengthening/retrofitting reinforced concrete (RC) columns. Although extensive research has been conducted on FRP-confined concrete columns under concentric compression, leading to many stress-strain models, the applicability of these concentric-loading (CL) stress-strain models in the analysis of columns under eccentric loading has not been properly clarified. This paper presents an in-depth investigation into this problem using an advanced three-dimensional (3D) finite element (FE) approach. The axial stress distributions and the stress-strain responses of FRP-confined concrete in circular columns under eccentric compression are examined using the FE results. It is shown that the stress-strain response of concrete varies greatly over the section and the direct use of a single CL stress-strain model for the entire section in the analysis of eccentrically-loaded columns may lead to significant errors, especially for the prediction of ultimate deformation/ductility. The paper also shows that a stress-strain model for the confined concrete at the extreme compression fibre of the section provides a relatively simple and much more accurate option for the analysis of eccentrically-loaded columns.

KEYWORDS

FRP, confinement, concrete, eccentric loading, column, stress-strain model, finite element analysis.

INTRODUCTION

Fiber-reinforced polymer (FRP) jacketing/wrapping has become a widely accepted technique for strengthening/retrofitting reinforced concrete (RC) columns (Teng et al. 2002; Hollaway and Teng 2008). To enable the safe and reliable design of FRP jackets, a large volume of research has been conducted on the behavior of FRP-confined concrete in these columns, leading to many uniaxial stress-strain models for FRP-confined concrete (Teng and Lam 2004). Most of these studies, however, have been focused on the behavior and modeling of FRP-confined concrete in columns subjected to concentric axial compression, where the concrete is subjected to (nominally) uniform confinement from the FRP jacket around the circumference. In reality, RC columns are rarely designed for concentric loading, and even when the intended loading is concentric, they are likely to be subjected to eccentric loading (i.e., combined compression and bending) as a result of unintended factors (e.g., accidental load eccentricity and geometric/material imperfections of the column).

Due to eccentric loading, a strain gradient (i.e., a linear strain variation) exists over the column section. A uniaxial stress-strain model established for FRP-confined concrete in columns under concentric loading [referred to as “concentric-loading (CL) stress-strain model” or simply “CL stress-strain model” hereafter] has generally been used in the analysis of RC columns under eccentric compression (e.g., Tao et al. 2004; Binici 2008; El Maaddawy 2008; Jiang 2008; Yuan et al. 2008; El Sayed and El Maaddawy 2011; Jiang and Teng 2012, 2013). The interest to find out whether a CL stress-strain model would work well for columns under eccentric loading and the need to quantify the effect of column slenderness have led to a number of experimental studies on FRP-confined concrete columns under eccentric loading (e.g., Fam et al. 2003; Li and Hadi 2003; Tao et al. 2004; Hadi 2006a, 2006b, 2007; Bisby and Ranger 2010; Hadi and Widiarsa 2012; Wu and Jiang 2013; Jiang et al. 2014). These experimental studies have shown that eccentrically-loaded columns exhibit some behavioral aspects that cannot be closely predicted using a CL stress-strain model. Based on the test results, several stress-strain models for FRP-confined concrete in columns under eccentric compression have been proposed (e.g., Fam et al. 2003; El Maaddawy 2009; Hu et al. 2011; Abdel Fattah 2012; Wu and Jiang 2013; Jiang et al. 2014). Such stress-strain models are generally eccentricity-dependent and are referred to as “eccentricity-dependent (EccD) stress-strain models” hereafter. A review of these stress-strain models has
shown that they predict significantly different or even opposite trends for the stress-strain behavior of FRP-confined concrete in columns under eccentric loading (Lin 2016). The lack of an in-depth investigation into the fundamental behavior of FRP-confined concrete in columns under eccentric loading due to the limitations of laboratory measurements is one of the main reasons for this controversy associated with the EccD stress-strain models.

This paper presents an improved understanding of the behavior of FRP-confined concrete (including axial stress distributions and stress-strain responses) in circular columns under eccentric compression based on results from an advanced three-dimensional (3D) finite element (FE) approach. The applicability of a CL stress-strain model for FRP-confined concrete in the analysis of columns under eccentric loading is given particular attention. The conclusions reached in this study will provide a solid basis for the development of a more rigorous and reliable EccD stress-strain model to enable more accurate predictions for FRP-confined RC columns under eccentric loading.

FE MODELING OF ECCENTRICALLY-LOADED COLUMNS

Lin and Teng (2016a) recently presented a 3D FE approach for modelling the behaviour of FRP-confined RC columns under eccentric loading and implemented it with ABAQUS (2011). In this approach, the whole column is considered in a 3D FE model (i.e., a full-height column model) although symmetry considerations should be taken advantage of to reduce the model size; for example, for a pin-ended column at both ends, only a quarter of the column need to be included in the FE model (Figure 1). The concrete, FRP jacket, and steel bars were represented respectively using 8-node solid elements (C3D8R), 4-node membrane elements (M3D4R), and 2-node truss elements (T3D2) available in ABAQUS (2011). Perfect bonding between the FRP jacket and the concrete was assumed, which was achieved by means of the “Tie Option” in ABAQUS (2011). The steel bars were also assumed to be perfectly bonded to the adjacent concrete using the “Embedded Option” in ABAQUS (2011) (Lin and Teng 2016a). The plastic-damage model developed by Yu et al. (2010a) with a small modification by Teng et al. (2015) was employed to model FRP-confined concrete. The FRP jacket was treated as an orthotropic linear-elastic-brittle material. For an FRP jacket with fibers only or mainly in the hoop direction of the column, the modulus of elasticity of the FRP jacket in the axial direction was assumed to have a very small value (e.g., 0.001 GPa). The FE approach has been shown to provide accurate predictions for many FRP-confined circular RC columns under eccentric loading (Lin and Teng 2016a).

Although a full-height column model is needed for the purpose of comparison with test results, such a model is not suitable for investigations into the effect of load eccentricity on the stress-strain behaviour of FRP-confined concrete. This is because the total actual eccentricity for a given section, which is equal to the sum of the initial eccentricity plus the additional eccentricity due to lateral deflections, varies with lateral deflections, which also means that the total actual eccentricity varies along the height of a deformed column. In order to isolate the total load eccentricity of a section as the only factor of variation, only a thin vertical slice of a column, representing the behaviour of a column section with a prescribed, constant load eccentricity, was modelled in the present study (Figure 2). The strain gradient (i.e., eccentric compression) was realized by imposing a monotonically increasing axial displacement that varies linearly across the column section to the nodes of the top surface of the slice model as shown in Figure 2. The nodes of the bottom surface of the slice model were restrained against displacements only in the vertical direction. Due to symmetry in geometry and loading, only half of the section needs to be included in the FE model (Figure 2). The location of the neutral axis (with a zero axial strain) was fixed during the loading process, which implies a varying combination of axial load and moment during the deformation process. By varying the location of the neutral axis, the stress-strain behaviour of FRP-confined

Figure 1 FE model for a typical column under eccentric compression

Figure 2 Slice model for a typical column under eccentric compression
CONFINEMENT MECHANISM UNDER ECCENTRIC LOADING

The results of the FE slice model for a typical FRP-confined circular concrete column are presented in this section. The column had a circular section with a diameter of 150 mm and was represented by a slice with a thickness of 5.0 mm. The nominal thickness, the elastic modulus, and the tensile rupture strain of the FRP jacket were 0.165 mm, 250 GPa, and 1.5%, respectively. The compressive strength of unconfined concrete was 30 MPa. Both the concrete and the FRP jacket had an element size of about 5.0 mm, chosen on the basis of a mesh convergence study (Lin and Teng 2016a). Diameter-to-compression depth ratios \( D/c \) ranging from 0.0 to 2.0 at an interval of 0.5 were examined (i.e., \( D/c = 0, 0.5, 1.0, 1.5 \) and 2.0). A \( D/c \) ratio equal to zero corresponds to concentric compression, and a larger \( D/c \) ratio represents a larger load eccentricity. The imposed displacement was monotonically increased until the hoop strain of the FRP jacket at the extreme compression fibre (ECF), \( \varepsilon_{h,c} \), reached the FRP tensile rupture strain \( \varepsilon_f \) (1.5%). This criterion was adopted based on the commonly accepted experimental observation that the recorded FRP rupture strain in an eccentrically-loaded column is close to the rupture strain from the standard FRP coupon tensile test (Lin and Teng 2016a).

Distributions of Axial Stresses

Figure 3 shows the axial stress distributions over the section with a \( D/c \) ratio of 1.5 when the hoop strain of the FRP jacket at the ECF, \( \varepsilon_{h,c} \), is at 0.1%, 0.5%, 1.0%, and 1.5%, respectively. The axial stresses are obviously non-uniform in the direction of bending (i.e., the X direction in Figure 3) due to the strain gradient, with the largest axial stress being at the ECF where the confinement from the FRP jacket is the highest. In the Y direction, the axial stresses are close to being uniform, which indicates that the axial stress distribution over the entire section can be well represented by the average axial stress distribution in the X direction only.

Average Axial Stress-Axial Strain Responses

Figure 4 shows the average axial stress-axial strain responses of concrete at nine selected locations evenly distributed in the X direction of the circular section. It can be seen that the axial stress-axial strain curves of concrete at locations 4 and 5, which are near the neutral axis, are well below the CL stress-strain curve even
though they are in the compression zone of the section. This is due to the stress state of combined compressive and tensile stresses in the section near the neutral axis, leading to a reduced axial stress compared to that under tri-axial compression (i.e., axial compression in combination with two equal lateral compressive stresses) (Lin and Teng 2016a). The CL stress-strain curve was obtained from the FE results for the section under concentric compression (i.e., $D/c = 0$), which is also identical to the stress-strain curve predicted by Jiang and Teng’s (2007) model for FRP-confined concrete that was used as a basis of the FE approach (Lin and Teng 2016a). The axial stress-axial strain curves of concrete near the ECF (at locations 6, 7, 8, and 9) generally feature a bilinear shape similar to the CL stress-strain curve. However, these axial stress-axial strain curves have a smaller slope for the linear second segment (referred to as the second-segment slope hereafter) but a larger ultimate axial stress and a larger ultimate axial strain than the CL stress-strain curve (Figure 4). The observation of a larger ultimate axial strain of concrete at the ECF of an eccentrically-loaded column than the ultimate axial strain of a corresponding concentrically-loaded column or that expected based on a CL stress-strain model has also been made in many experimental studies (i.e., the axial strain enhancement effect) (e.g., Scott et al. 1982; Fam et al. 2003; Bissy and Ranger 2010; Yu et al. 2010; Csuka and Kollár 2012; Wu and Jiang 2013). This is because the expansion of concrete over an eccentrically-loaded section is non-uniform with the concrete in the vicinity of the ECF having larger expansions due to larger axial strains there than the rest of the concrete. Therefore, the axial strain of concrete at the ECF in an eccentrically-loaded section to induce the same FRP hoop strain there needs to be larger than that for a concentrically-loaded section.

As the concrete at the ECF is subjected to the highest axial stress over the column section and has the greatest distance from the neutral axis, it is believed that its stress-strain response plays a key role in determining the response of the entire section. Figure 5 shows the stress-strain curves of concrete at the ECF of sections with different $D/c$ ratios from FE analysis. It is obvious that the second-segment slope decreases but the ultimate axial strain increases as the $D/c$ ratio increases, with the combined effect being a slight increase in the ultimate axial stress.

![Figure 4 Average stress-strain responses of concrete at different locations ($D/c = 1.5$)](image)

![Figure 5 Stress-strain curves of concrete at ECF of sections with different $D/c$ ratios](image)

![Figure 6 Axial load-curvature curves obtained with different stress-strain curves](image)

![Figure 7 Interaction diagrams obtained with different stress-strain curves](image)
EFFECT OF STRESS-STRAIN CURVE ON COLUMN RESPONSE

The above discussion indicates that the direct use of a CL stress-strain model in the analysis of columns under eccentric loading may lead to unreliable results. In order to clarify this effect, the responses of sections with different $D/c$ ratios were predicted using section analysis based on two different stress-strain curves: one is the CL stress-strain curve and the other is the stress-strain curve of the concrete at the ECF from FE analysis. In the section analysis, the section was divided into 50 layers in the direction of bending (i.e., in the X direction) and all concrete layers were assigned with the same stress-strain curve (Lin and Teng 2016a).

The axial load-section curvature curves for sections with different $D/c$ ratios from section analysis are compared with FE results in Figure 6. It can be seen that the predictions with the stress-strain curve of concrete at the ECF are much closer to the FE results than those with the CL stress-strain curve. The ultimate curvatures obtained using the CL stress-strain curve are generally much smaller than the FE results. It is worth noting that the axial load-section curvature curves from section analysis are not exactly the same as the FE curves, and this is due to errors caused by approximating the stress-strain responses of concrete over the whole section by that of a single stress-strain curve for the ECF in section analysis. The axial load-moment interaction diagrams from section analysis and that from FE analysis are compared in Figure 7. It can be seen that these diagrams are close to each other, indicating that the strain gradient effect on the stress-strain response has only a small bearing on the load-carrying capacity of sections under eccentric loading.

ECCENTRICITY-DEPENDENT STRESS-STRAIN MODEL

Based on the above observations, a stress-strain model for the concrete at the ECF instead of a CL stress-strain model should be developed for use in the section analysis of eccentrically-loaded FRP-confined concrete columns as a relatively simple option for reasonably accurate predictions, especially for the ultimate deformation/ductility. As clearly indicated in Figure 5, the second-segment slope of the stress-strain curve of concrete at the ECF decreases but the ultimate axial strain increases as the $D/c$ ratio increases. Based on these trends, an eccentricity-dependent (EccD) stress-strain model can be established by modifying a CL stress-strain model if the relationship between the two models can be established. A comprehensive parametric study using the aforementioned 3D FE slice model was conducted by the authors, with all the parameters that affect the responses of FRP-confined circular sections under eccentric compression being properly examined (Lin and Teng 2016b). The results of the parametric study allowed the relationship between the stress-strain behaviour of concrete at the ECF and the corresponding CL stress-strain model to be clarified over a wide range of section parameters, based on which an EccD stress-strain model was established (Lin and Teng 2016b). This EccD stress-strain model can be directly used in section analysis of eccentrically-loaded FRP-confined RC columns. The proposed EccD stress-strain model has been shown to provide much more accurate predictions for the test results of eccentrically-loaded FRP-confined circular RC columns than the corresponding CL stress-strain model (Lin and Teng 2016b).

CONCLUSIONS

This paper has presented an improved understanding of the behaviour of FRP-confined concrete in a circular section under eccentric compression obtained from a 3D FE investigation. The following conclusions can be drawn from the results and discussions presented in the paper:

1. The uniaxial stress-strain response of concrete varies greatly over the section and may differ greatly from a stress-strain model obtained from studies on FRP-confined concrete columns under concentric loading (CL stress-strain model);
2. In the vicinity of the extreme compression fibre (EFC) of the section, the concrete receives much stronger confinement than the concrete near the neural axis, and has a bilinear stress-strain curve similar to that of a CL stress-strain model;
3. The direct use of a CL stress-strain model in the section analysis of an eccentrically-loaded column may lead to significant errors, especially for the prediction of ultimate deformation/ductility;
4. A stress-strain model for the concrete at the ECF offers a good option as the stress-strain model for section analysis of such columns for reasonably accurate predictions;
5. As the $D/c$ ratio increases, the second-segment slope of the stress-strain curve of concrete at the ECF decreases but the ultimate axial strain increases. Based on these trends, a so-called eccentricity-dependent
(EccD) stress-strain model can be established for the analysis of eccentrically-loaded columns by modifying an existing CL stress-strain model.

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EXPERIMENTAL STUDY ON BFRP CONFINED RAC STUBS UNDER CYCLIC AXIAL COMPRESSION

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ABSTRACT

Reusing concrete can not only save natural resources, but also develop the composites of recycled aggregate concrete (RAC), among which are the fibre-reinforced polymer (FRP) confined RAC columns. The experiment for relatively large scale (with a diameter of 300mm) of prefabricated basalt fibre-reinforced polymer (BFRP) tube confined RAC stub columns (BCRSs) were performed under cyclic axial compression. Three parameters, including the strength of RAC, the recycled coarse aggregate (RCA) replacement ratio and the loading rate, were set to study the mechanical properties of these composites. The curves of Load-displacement, the envelopes of stress-axial strain, the curves of stress-hoop strain were obtained. The residual strain and stress degradation were analyzed, and a modified effective confinement factor of BCRSs was proposed. The experimental results will be a good supplement for the data base of FRP confined concrete columns.

KEYWORDS

Basalt fibre-reinforced polymer (BFRP), recycled aggregate concrete (RAC), cyclic axial compression, residual strain, stress degradation, effective confinement factor.

INTRODUCTION

As a form of construction garbage regeneration, recycled aggregate concrete (RAC) has been newly reused into construction (Rao et al. 2007; Silva et al. 2014) due to its social, environmental and economic significance. However, it generally restricted to non principal components with low load-bearing capacity because of its originally high bibulous rate, large porosity and low strength (Al-Bayati et al. 2016; Fan et al. 2016; Kovler and Roussel 2011). Existed researches have shown that the performance of concrete members can be significantly enhanced by external confinement using fiber-reinforced polymer (FRP) (Xiao et al. 2012; Yu et al. 2015), due to its superior material properties, such as high tensile strength to weight ratio and excellent corrosion resistance (Wei et al. 2009).

So far, numerous experimental studies have been conducted to investigate the mechanical properties of components confined with high-tensile-strength carbon fiber-reinforced polymer (CFRP) or high-ductility glass fiber-reinforced polymer (GFRP) (Wu and Wei 2010; Seffo and Hamcho 2012; Almusallam 2007), but very few have related to basalt fiber reinforced polymer (BFRP), which has high cost efficiency and promising application prospect (Ozbakkaloglu 2013). Moreover, most studies have limited to focus on FRP confined RAC columns under monotonic axial loading (Jiang et al. 2016; Chastre and Silva 2010). This paper presents an experiment performed on BFRP confined RAC stub columns (BCRSs) under cyclic axial compression, which may be essential to seismically active areas.

EXPERIMENT PROGRAM

8 specimens of BCRSs were tested under cyclic axial compression with a diameter of 300mm and a height of 600mm. Three parameters were included, which were recycled coarse aggregate (RCA) replacement rates (0%, 30%, 50%, 70% and 100%), the strength of RAC (C40 and C60) and the loading process (Type A and Type B). The specimens were listed in Table 1. Each specimen was prepared 3 cubes and 3 cylinders respectively to test the concrete strength.

Materials and Specimen Preparation
BFRP tubes were prefabricated with an inner diameter of 300mm and a thickness of (2.35±0.25)mm by 40% of basalt fibre spirals in 80°. All pipes used were divided from the same BFRP tube after cutting the length of 500mm at both sides. The ultimate tensile strength of BFRP tube was 690MPa and modulus of elasticity was 51.3 GPa according to the measurement at laboratory.

Table 1 Main parameters of specimens

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<th>Group</th>
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<th>RAC strength/MPa</th>
<th>RCA replacement rate/%</th>
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<td>A</td>
<td>Cylinder</td>
<td>55.320</td>
</tr>
<tr>
<td>4</td>
<td>C40-R70-A</td>
<td>40</td>
<td>70</td>
<td>A</td>
<td>Cylinder</td>
<td>53.346</td>
</tr>
<tr>
<td>5</td>
<td>C40-R100-A</td>
<td>40</td>
<td>100</td>
<td>A</td>
<td>Cylinder</td>
<td>54.966</td>
</tr>
<tr>
<td>6</td>
<td>C60-R50-A</td>
<td>60</td>
<td>50</td>
<td>A</td>
<td>Cylinder</td>
<td>76.635</td>
</tr>
<tr>
<td>7</td>
<td>C60-R100-A</td>
<td>60</td>
<td>100</td>
<td>A</td>
<td>Cylinder</td>
<td>75.805</td>
</tr>
<tr>
<td>8</td>
<td>C60-R100-B</td>
<td>60</td>
<td>100</td>
<td>B</td>
<td>Cylinder</td>
<td>75.805</td>
</tr>
</tbody>
</table>

Two grades of core concrete strength was C40 and C60. The lower strength concrete mixture ratio was cement: water: fine aggregate: coarse aggregate = 225: 475:641:1057 with a water cement ratio of 0.47, while the other mixture ratio was cement: water: fine aggregate: coarse aggregate: water reducing agent = 214: 643: 395: 1058.5: 10.28 with a water-cement ratio of 0.35. The fine aggregate was natural river sand with a fineness modulus of 2.4. The RCA was produced in the manufacture from a served building for over twenty years. Its properties were listed in Table 2, as well as the natural coarse aggregate (NCA).

The BFRP tube was served as a template for concrete, which was cast in the laboratory. The RCA replacement rate is based on the volume of coarse aggregate.

Table 2 Properties of NCA and RCA

<table>
<thead>
<tr>
<th>Aggregate</th>
<th>Partical size /mm</th>
<th>Apparent density /kg·m⁻³</th>
<th>Bulk accumulation density/kg·m⁻³</th>
<th>Water absorption (%)</th>
<th>Crush index (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NCA</td>
<td>5~25</td>
<td>2730</td>
<td>1480</td>
<td>2.36</td>
<td>5.24</td>
</tr>
<tr>
<td>RCA</td>
<td>5~25</td>
<td>2510</td>
<td>1393</td>
<td>3.10</td>
<td>10.62</td>
</tr>
</tbody>
</table>

Test Setup and Measurements

For each BCRS, two bi-directional strain gauges with a length of 20mm were installed at the mid-height spaced 90° around the outer surface of the BFRP tube, totally 8 strain gauges. 4 linear variable displacement transducers (LVDTs) with a range of 50mm were additionally used to obtain the axial deformation for the whole column. All test data, including the strains, loads and displacements, were recorded simultaneously by a data logger, TDS530 static measurement acquisition system. The test setup and instruments were shown in Figure 1.
All cyclic axial compression tests were carried out using an electric-fluid servo compression machine with a loading maximum of 10000kN. The whole loading was controlled by displacement. Two loading types (Type A and Type B) were both designed as mixed loading schemes. The type A with a loading rate of 0.36mm/min involves: (1) 1 single full unloading/reloading (load below 70kN) cycle at displacement value of 0.6mm; (2) 11 repeated full unloading/reloading cycles, whose unloading displacement value was 0.6mm larger than the former one; (3) 6 repeated full unloading/reloading cycles at the displacement value of 7.8mm (0.6mm×13); (4) up to the specimen fracture. The type B with a loading rate of 0.54mm/min involves: (1) 1 single full unloading/reloading cycle at displacement value of 0.6mm; (2) 11 repeated full unloading/reloading cycles, whose unloading displacement value was 1.2mm larger than the former one until the specimen fracture which appeared the bearing capacity of BCRSs dropped dramatically below 50% of the peak load.

RESULTS AND DISCUSSIONS

Failure Modes

Specimens after test were shown in Figure 2. The failure modes of BCRSs revealed an explosive rupture of BFRP jacket in the mid-height and a little above region, which were similar with that of FRP confined NAC members under monotonic axial compression. As to the BCRSs with low-strength RAC, the filamentous fracture of the outer tubes could be seem, following with concrete smashing, which had a certain predictability before destruction. However, for the BCRSs with high-strength RAC, the BFRP tubes were generally laminar tearing and concrete was splitting aslant without any predictability.

![Figure 2 Failure modes of specimens](image)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Peak load /kN</th>
<th>Peak displacement/mm</th>
<th>Ultimate load/kN</th>
<th>Ultimate displacement/mm</th>
<th>Equivalent concrete strength /MPa</th>
<th>BCRS strength /MPa</th>
<th>Enhancing coefficient</th>
<th>Effective confinement factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>C40-R0-A</td>
<td>4412</td>
<td>11.355</td>
<td>4412</td>
<td>11.355</td>
<td>43.204</td>
<td>62.417</td>
<td>1.45</td>
<td>2.37</td>
</tr>
<tr>
<td>C40-R30-A</td>
<td>4013</td>
<td>10.815</td>
<td>4013</td>
<td>10.815</td>
<td>41.956</td>
<td>56.772</td>
<td>1.35</td>
<td>1.83</td>
</tr>
<tr>
<td>C40-R50-A</td>
<td>4519</td>
<td>12.565</td>
<td>4519</td>
<td>12.565</td>
<td>41.328</td>
<td>63.931</td>
<td>1.55</td>
<td>2.79</td>
</tr>
<tr>
<td>C40-R70-A</td>
<td>4076</td>
<td>11.633</td>
<td>4076</td>
<td>11.633</td>
<td>41.649</td>
<td>57.664</td>
<td>1.39</td>
<td>1.98</td>
</tr>
<tr>
<td>C60-R50-A</td>
<td>4934</td>
<td>3.011</td>
<td>4550</td>
<td>9.438</td>
<td>60.673</td>
<td>69.802</td>
<td>1.15</td>
<td>1.13</td>
</tr>
<tr>
<td>C60-R100-A</td>
<td>4957</td>
<td>3.005</td>
<td>4665</td>
<td>9.75</td>
<td>60.329</td>
<td>70.127</td>
<td>1.16</td>
<td>1.21</td>
</tr>
<tr>
<td>C60-R100-B</td>
<td>4586</td>
<td>10.865</td>
<td>4586</td>
<td>10.865</td>
<td>60.329</td>
<td>64.879</td>
<td>1.08</td>
<td>0.56</td>
</tr>
</tbody>
</table>

Bearing Capacity

The load-deformation curves for some specimens were shown in Figure 3. The main test results were summarized in Table 3, where the enhancing coefficient of bearing capacity defined as the ratio of BCRS strength to equivalent concrete strength from cylinders (Sabins and Mirza 1979). From the enhancing coefficient, it can be seen that the BFRP confinement improve the load-bearing capacity of low-strength RAC effectively by approximately 50%, even reaching 62% when RCA replacement ratio is 100%. However, the improvement of the BCRSs with high-strength concrete was less than 20%, which indicates that cyclic axial compression has obvious negative influence on the high-strength BCRSs.

The strength model of FRP confined concrete is defined by the following Eq. 1 (Teng, J.G, 2002).

\[
\frac{f}{f_{\text{c}}} = 1 + k \cdot \frac{f}{f_{\text{c}}}
\]

(1)
where $f_c$ and $f'_c$ are the compressive strength of confined concrete and unconfined concrete respectively; $k_1$ is the effective confinement factor; and $f_l$ is the lateral restraint stress, which is defined by Eq. 2 (Teng, J.G et al. 2002).

$$f_l = \frac{2f_{frp}d}{d}$$

where $f_{frp}$ and $t_{frp}$ are the hoop tensile strength and thickness of FRP respectively; and $d$ is the diameter of confined concrete.

It should be noted that the effective confinement factor can be applied for BCRSs under cyclic axial compression, and the value is suggested as 1.8 for the BCRSs with low-strength concrete, but no longer applicable for the BCRSs with high-strength concrete.

![Figure 3 Curves of load-deformation](image)

![Figure 4 Envelopes of stress-axial strain](image)

![Figure 5 Curves of residual strain-unloading strain](image)

**Envelopes of Stress-Strain**

The envelopes of stress $\sigma$–axial strain $\varepsilon_v$ were shown in Figure 4. The curves of the BCRSs with low-strength concrete have an obvious enhancing section, in which the bearing capacity and ductility are improved significantly. The RCA replacement rate has little effect on the mechanical properties of BCRSs. The load-bearing capacity and the deformation ability of BCRSs with RAC replacement ratios of 50% and 100% are both better than that with...
natural concrete. The BCRS with a rate of 30% is similar with the specimen with a rate of 70% in strength and ductility. Compared with the low-strength ones, the first section of the high-strength BFRSs’ curves has a larger slope, which drops slightly after reaching the second section whose slope is nearly zero. The ductility of high-strength BFRSs is not as good as that of low-strength ones. For higher cyclic loading speed, the load-bearing capacity of BFRSs decreases, but a slight increase in ductility.

Residual strain

The last loading has an accumulation effect on deformation for the next loading after the period of elasticity, thus there will be a residual strain, or permanent deformation, after unloading. The relationship of residual strain $\varepsilon_{re}$ and unloading strain $\varepsilon_{un}$ were shown in Figure 5. The relationship can be delivered through the linear function fitting, as Eq. 3, with an adjust determination coefficient of 0.98949.

$$\varepsilon_{re} = 0.72717 \varepsilon_{un} - 0.00115 \quad (3)$$

Hoop Strain

Two of the stress $\sigma$-hoop strain $\varepsilon_h$ curves were shown in Figure 6. The development of its relationship is the same as that of the stress and axial strain.

Stress degradation

The load-bearing capacity of specimens would be lower after reloading to the same displacement compared to last reloading, which is measured by stress degradation rate. The primary stress $\varphi_1$ is the ratio of stress $\sigma_{un,1}$ in the second reloading corresponding with the first unloading strain $\sigma_{un}$ to the first unloading stress. The $n^{th}$ stress degradation rate $\varphi_n$ is the ratio of stress $\sigma_{un,n}$ in the next reloading corresponding with the last unloading strain to the last unloading stress $\sigma_{un,n-1}$ in the last 6 repeated loading circle. The stress degradation was shown in Figure 7. In Figure 7a, the values of primary stress are ranged from 0.85 to 1. The primary stress decreases before the unloading strain approximately reaching to 0.004, after which develops steadily. However, the discretion is larger for the BCRSs with high-strength concrete, especially at a high loading rate. From Figure 7b, the value of $n^{th}$ stress degradation rate is ranged from 0.92 to 1, without large deviation among specimens.
CONCLUSIONS

A cyclic axial compression test was performed on BCRSs with different RCA replacement ratios, concrete strength and loading rates. The following test results could be concluded.

1. The low-strength BCRSs were shown the filamentous fracture of the outer tubes and concrete smashing, while the high-strength ones were presented BFRP tubes laminar tearing and concrete splitting aslant without any predictability.

2. BCRSs improve the load-bearing capacity of low strength concrete effectively by approximately 50%, while the improvement of high-strength BCRSs was less than 20%. The effective confinement factor can be applied for BCRSs under cyclic axial compression, and the value is suggested as 1.8 for the low-strength BCRSs.

3. The envelopes of stress-strain for the low-strength BCRSs has an obvious enhancing section. The RCA replacement ratio has little effect on the mechanical properties of BCRSs. The ductility of high-strength BFRSs is not as good as that of low-strength ones. For higher cyclic loading speed, the bearing capacity of BFRSs decreases, but a slight increase in ductility.

4. The relationship of residual strain and unloading strain can be delivered through the linear function fitting.

5. The primary stress degradation rate develops steadily after unloading strain approximately reaching to 0.004. The value of n° stress degradation rate is ranged from 0.92 to 1, without large deviation.

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REFERENCES


PROBABILISTIC ASSESSMENT OF STRENGTH AND DUCTILITY OF FRP-CONFINED CONCRETE COLUMNS

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ABSTRACT
Confinement using Fibre-reinforced polymers (FRP) significantly improves both the strength and the ductility of reinforced concrete columns. Many analytical models for prediction of strength and ductility of the FRP-confined concrete can be found in the literature. However, most of these models were developed based on conventional deterministic approaches. In this study, a probabilistic approach for predicting the ultimate strength and strain of FRP-confined concrete is developed. Using probabilistic models for concrete and FRP materials and the Monte Carlo simulation technique, uncertainty in evaluating the ultimate strength and strain in FRP-confined concrete is investigated and lower percentile design models are suggested.

KEYWORDS
Reliability, FRP-confined concrete, strength, ductility, design models.

INTRODUCTION
RC columns serve an important role in the load-carrying capacity of concrete structures. Failure of a column, due to overloading or age deterioration, would lead to more catastrophic consequences than would the failure of other structural elements. Therefore, it is necessary to strengthen deteriorated or deficient RC columns to increase their load-carrying capacity, and to improve their ductility in order to better their performance against accidental and seismic loads. In the last decade or so, the retrofit of RC columns by wrapping them with FRP sheets has gained increasing interest in the engineering community (Binici 2008; Demers and Neale 1994; Ozcan et al. 2010; Saadatmanesh et al. 1994). The work of many researchers has indicated that FRP confinement (especially in the potential plastic hinge regions) can substantially enhance the axial compressive strength and ductility of RC columns (Binici 2008; Gu et al. 2012; Ozbakkaloglu and Saatcioglu 2006), and this is an important rehabilitation option to consider. The effectiveness of FRP confinement in increasing the strength and ductility is attributed to its ability to avoid or delay undesirable phenomena such as concrete crushing or longitudinal reinforcing bar buckling.

Accurate estimation of the ductility and strength of FRP-confined columns is necessary for a reliable retrofit strategy. Due to the inherent uncertainties in material properties and geometrical dimensions, structural strength and ductility exhibit variability, which needs to be statistically quantified. Furthermore, there is always uncertainty associated with the models used for predicting structural responses. Therefore, a reliable predictive model should result from a probabilistic procedure containing all the mentioned uncertainties. The probabilistic procedure for deriving predictive design models involves four important steps: (i) developing a best-fit predictive model; (ii) establishing a probabilistic model for the model error; (iii) establishing/collecting statistical models for the main random variables, such as material properties; and (iv) developing a probabilistic design model based on a specific confidence level. The probabilistic models can then be used to predict the axial/flexural strength and ductility of FRP-confined columns or to investigate the reliability of FRP-confined columns. In any development of design provisions for FRP-confined concrete members, probabilistic models for the ultimate strength and strain of FRP-confined concrete are essential. These models can directly be used for calculating the design values of the ultimate strength and strain of FRP-confined concrete. They can also be useful in any moment-curvature analysis aimed at predicting the design curvature ductility and flexural strength of columns. Most importantly, these models can be employed in reliability-based calibration of resistance reduction factors.
This paper presents a probabilistic procedure for evaluating the ultimate strength and strain of FRP-confined circular columns. A probabilistic procedure based on the Monte Carlo simulation is employed for establishing lower percentile design models for the ultimate strength and strain. The methodology presented here can be extended to columns with rectangular or square sections.

FORMULATION OF STRENGTH AND DUCTILITY MODELS

Mechanism of FRP confinement

Figure 1 illustrates the stress state of a circular concrete column wrapped with FRP. Using the force equilibrium and considering the linearity of FRP materials behaviour, the lateral (radial) confining pressure acting on the concrete core at the FRP jacket failure at the ultimate state, \( f_r \), is given by

\[
f_r = \frac{2\sigma_{fu} t_f}{D} = \frac{2E_f \varepsilon_{fu} t_f}{D}
\]

where \( \sigma_{fu} \) and \( t_f \) are the FRP ultimate strength and thickness of the FRP jacket, respectively, and \( D \) represents the diameter of the concrete cylinder. Variables \( E_f \) and \( \varepsilon_{fu} \) denote the Young's modulus and the ultimate radial strain of the FRP jacket. In Eq. 1, by assuming a perfect bond between the FRP jacket and the concrete core, the radial strain in the FRP membrane is equal to that of the concrete core.

![Figure 1 Stress diagrams in FRP-confined concrete](image)

The confining pressure around the concrete core increases proportionally with the FRP hoop strain, until the point when the whole system fails due to the failure of the FRP fibres. Thus, at the failure state, Eq. 1 can establish a relationship between the ultimate lateral pressure, \( f_r \), and the rupture strain of the FRP jacket, \( \varepsilon_{f,rup} \). The failure is characterised by the tensile fracture of the FRP jacket. However, it has been observed that, at failure, the tensile strength of the FRP wrap is different from the uniaxial tensile strength of the FRP (Lam and Teng 2003; Pessiki et al. 2001). To address the difference between the actual rupture strain and the ultimate strain of FRP material, by replacing the FRP ultimate tensile strain, \( \varepsilon_{fu} \), with the actual hoop rupture, \( \varepsilon_{f,rup} \), the actual confining pressure, which is here denoted as \( f_{r,rup} \), and can be seen in Eqs. 2a and 2b, is introduced.

\[
e_{f,rup} = k_e \varepsilon_{fu}
\]

\[
f_{r,rup} = \frac{2E_f \varepsilon_{f,rup} t_f}{D}
\]

Results from different tests show that the so-called FRP strain efficiency factor, \( k_e \), given by the ratio \( \varepsilon_{f,rup} / \varepsilon_{fu} \), varies for different types of FRP materials.

Another term that is usually used regarding FRP-confined concrete is the so-called “jacket lateral stiffness”, which is here denoted using the symbol \( E_j \). Using Eq. 1, and taking advantage of Hook’s law, Eqs. 3a and 3b result.

\[
f_r = \frac{2E_f \varepsilon_{fu} t_f}{D} = E_j \varepsilon_{fu}
\]

\[
E_j = \frac{2E_f t_f}{D}
\]

The jacket lateral stiffness (also called jacket lateral modulus) plays an important role in the derivation of expressions for the confined concrete dilation rate.
Strength model

In this study, a linear model is used to predict the ultimate strength of FRP-confined columns, $f'_{cc}$. The linear model used in this study follows the original form suggested by Richart et al. (1928), based on the well-established Mohr-Coulomb failure criterion. If a least-square regression analysis is used to fit the data to a line relating the axial and the lateral stresses, as is shown in Figure 2a, the slope of this line represents the confinement effectiveness coefficient, $k$. Eq. (4) shows this approach for finding this factor.

$$\frac{f'_{cc}}{f'} = 1 + k \frac{f'_{tu}}{f'_r} = 1 + k \frac{2k'\lambda_{tu}}{Df'_r}$$  \hspace{1cm} (4)

In Eq. (4), $f'_{cc}$ is the maximum strength of FRP-confined concrete. It should be noted that in the regression analysis that leads to the determination of $k$, the fitting line is forced to pass through the point representing the uniaxial test results of plain unconfined concrete. In the model proposed by Richart et al. (1928), a confinement effectiveness factor of 4.10 was suggested for actively confined concrete. For FRP-confined concrete, based on the model proposed by Lam and Teng (2003), this factor is 3.30. This model will be adopted in the probabilistic analysis presented in the next section.

Ductility model

In this study, a simplified model initially proposed by Tamuzs et al. (2006) and later used by Berthet et al. (2006) for predicting the ultimate strain of concrete, which can be seen in Figure 2b, is adopted. In this model, using a bilinear curve (as shown in Figure 2b), the dilation behaviour of FRP-confined concrete is approximated. The initial slope, which represents the elastic behaviour, is the Poisson ratio, while the second slope, which represents the post-cracking behaviour, is the asymptotic plastic dilation ratio. Berthet et al. (2005) showed that, for specimens experiencing strength enhancement, this bilinear representation is a reasonable approximation and it would considerably simplify the derivation of the ultimate strain of FRP-confined concrete. A recent study by Ozbakkaloglu et al. (2013) showed that predictive models based on this form of approximation are more accurate than other types of models. It has been reported that the asymptotic values of the dilation ratio and dilation rate, $\eta_u$ and $\mu_u$, are approximately equal (Tamuzs et al. 2006). Referring to Figure 2b, the ultimate strain of FRP-confined concrete, $\varepsilon_{cc}$, can be calculated as shown in Eq. 5.

$$\varepsilon_{cc} = 1.00 + \left( \frac{1}{\mu_u} \right) \left( \frac{\varepsilon_{f,upp} - \varepsilon_{c,0}}{\varepsilon_{c,0}} \right)$$  \hspace{1cm} (5)

The strain ratio in Eq. 5 can be considered as the strain enhancement or ductility. Since the dilation behaviour of confined concrete depends on the confinement level, as an indicator of volumetric dilation, the dilation rate, $\mu_u$, could reasonably be related to the properties of the FRP jacket and, in particular, to the jacket lateral stiffness. A model proposed by Baji et al. (2016), which can be seen in Eq. 6b, is adopted for calculating the ultimate dilation rate, $\mu_u$.

PROBABILISTIC APPROACH

According to the developed analytical models, the ultimate strength and strain enhancements are shown in Eqs. 6a and 6b. In comparison with the total ultimate strain, the elastic lateral strain i.e $\nu_0$ (see Eq. 6b) is negligible.
Therefore, in the ductility model, shown in Eq. 6b, it is ignored. As can be seen, the second terms of the both enhancement models are the products of different random variables.

\[
S = f'_{c} - 1.00 = \xi (3.30) \left( \frac{2k_{c} \sigma_{n}}{D'_{c}} \right) \tag{6a}
\]

\[
M = \frac{e_{c0}}{\xi_{c0}} - 1.00 = \xi \left( \frac{1}{5.1} \right) \left( \frac{k_{c} \sigma_{n}}{E_{r} e_{c0}} \right) \left( \frac{2E_{r} f'_{c}}{D'_{c}} \right)^{0.36} \tag{6b}
\]

where \( \xi \) represents the model error. The normalised strength enhancement \( (S) \) and strain enhancement \( (M) \), can be treated as random variables. Using a probabilistic study, statistics of these variables can be derived. Furthermore, the lower percentile values can be calibrated as design models. For developing these design models, probabilistic models for all the variables shown in Eqs. 6a and 6b are required. Typical probabilistic models taken from the current literature, shown in Table 1, will be used in this study. The nominal value for the FRP strain efficiency factor is taken from the ACI 440 (2008).

<table>
<thead>
<tr>
<th>Variable</th>
<th>Nominal</th>
<th>Statistics</th>
<th>Distribution</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>( D (\text{mm}) )</td>
<td>150</td>
<td>( \lambda = 1.00 )</td>
<td>0.04</td>
<td>Normal</td>
</tr>
<tr>
<td>( f'_{c} (\text{MPa}) )</td>
<td>20-80</td>
<td>( \mu = f'_{c} + 8 )</td>
<td>( \sigma = 6 )</td>
<td>Lognormal</td>
</tr>
<tr>
<td>( e_{c0} )</td>
<td>0.002</td>
<td>( \lambda = 1.30 )</td>
<td>( V = 0.18 )</td>
<td>Lognormal</td>
</tr>
<tr>
<td>( t_{f} (\text{mm}) )</td>
<td>1.20</td>
<td>( \lambda = 1.00 )</td>
<td>( V = 0.05 )</td>
<td>Lognormal</td>
</tr>
<tr>
<td>( \sigma_{fu} (\text{MPa}) )</td>
<td>887.1</td>
<td>( \lambda = 1.17 )</td>
<td>( V = 0.15 )</td>
<td>Weibull</td>
</tr>
<tr>
<td>( E_{f} (\text{MPa}) )</td>
<td>64900</td>
<td>( \lambda = 1.10 )</td>
<td>( V = 0.20 )</td>
<td>Lognormal</td>
</tr>
<tr>
<td>( k_{c} )</td>
<td>0.55</td>
<td>( \lambda = 1.23 )</td>
<td>( V = 0.30 )</td>
<td>Weibull</td>
</tr>
<tr>
<td>( \xi_{d} )</td>
<td>1.00</td>
<td>( \lambda = 1.00 )</td>
<td>( V = 0.37 )</td>
<td>Lognormal</td>
</tr>
<tr>
<td>( \xi_{d} )</td>
<td>1.00</td>
<td>( \lambda = 1.07 )</td>
<td>( V = 0.36 )</td>
<td>Weibull</td>
</tr>
</tbody>
</table>

\( \mu = \text{mean}, \lambda = \text{bias factor}, V = \text{coefficient of variation} \)

The lower percentile value of the ultimate strength and strain can be used for design and code calibration purposes. In comparison with the conventional deterministic models the lower percentile design models, such as models presented in this study, provide more consistent level of safety in the design procedure. The resistance predicted by a design model is lower than that of deterministic model, which is based on nominal values of the variables. In this research, a relationship between the nominal ultimate strength and strain enhancements and design values (based on lower percentile) of these variables is established. This relationship considers uncertainties in all the main random variables. Derivation of the ratio of lower percentile to nominal values based on the empirical cumulative probability density function of the strength or ductility enhancement is graphically shown in Figure 3. The subscripts \( p \) and \( n \) denote the lower percentile value and the nominal values, respectively. The Monte Carlo simulation technique (Melchers 1999) is used for finding the empirical cumulative probability density function of both the strength and strain enhancements.

The lower percentile value of the ultimate strength and strain can be used for design and code calibration purposes.

![Figure 3 Probabilistic lower percentile values i.e. design models](image_url)
RESULTS AND DISCUSSIONS

Following the solution provided in the previous section, the ratio of the $p^{th}$ percentile to the nominal value, of the strength and strain enhancement value can be calculated. The statistical models shown in Tables 1 can be used as an application example for the proposed model. Value of the lower percentile, $p$, is generally decided by code authorities and depends on the overall reliability analysis of the member as well. Generally, 5% and 10% are common in design practice. Based on the different lower percentile values ($p = 0.05, 0.10$ and $0.25$), variation of lower percentile to nominal ratio with respect to concrete compressive strength is shown in Figure 4.

Due to the dependence of statistical measures of concrete compressive strength on the concrete specified strength, the probabilistic model for the strength enhancement varies with change in the concrete compressive strength. Nevertheless, as shown in Figure 4, the variation is not considerable. For concrete compressive strengths above 30 MPa, the ratio of $S_{p=0.10} / S_n$ is about 0.60. Considering this ratio, the lower percentile strength model can be proposed as follows.

$$
\left[ \frac{f'_c}{f_{c0}} \right]_{p=0.10} = 1.00 + (0.60) \left[ \frac{2k \sigma_{sf \ell} \ell}{Df_c} \right]_{\text{Nominal}}
$$

(7)

Accuracy of the probabilistic-based model shown in Eq. 7 depends on the accuracy of the probabilistic models for the material properties of concrete and FRP and, more importantly, on the accuracy of the model error. Results for different $p$ values can be easily obtained using the established probabilistic procedure.

The strain enhancement or ductility variable ($\varepsilon_c / \varepsilon_{c0} - 1$) is a product of eight different random variables (shown in Eq. 6b). In comparison with the strength model, more random variables are involved in the strain model. The procedure used for deriving the strength enhancement model can be followed for obtaining the $p^{th}$ percentile value of the strain enhancement. The probabilistic models shown in Table 1 can again be used as a case study for calculating the statistics of the strain enhancement ratio and the $M_{p=0.10} / M_n$ ratio. Similar to the strength model, this ratio is nearly constant (about 0.45) for concrete compressive strengths of more than 30 MPa. Therefore, the $10^{th}$ percentile value model for the strain enhancement can be expressed as follows.

$$
\left[ \frac{\varepsilon_c}{\varepsilon_{c0}} \right]_{p=0.10} = 1.00 + (0.45) \left[ \frac{1}{5.1} \left( \frac{k \sigma_{sf \ell} \ell}{E_j \varepsilon_{c0}} \right)^{0.56} \right]_{\text{Nominal}}
$$

(8)

This model, along with the strength model shown in Eq. 7, can be used for the evaluations of the design strength and strain ductility.

CONCLUSIONS

Probabilistic investigation into the ultimate strength and strain of FRP-confined concrete was the main target of the current research. Analytical models for the ultimate strength and strain of FRP-confined concrete were taken from the available literature. Considering the variability in the model error, in the mechanical properties of concrete and FRP materials and using the Monte Carlo simulation technique, a probabilistic procedure for deriving design
models for the ultimate strength and strain was proposed. Design models based on a predefined lower percentile values were suggested.

ACKNOWLEDGMENTS

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ACI 440 (2008). "Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures." American Concrete Institute, Farmington Hills, MI, USA.


ABSTRACT

This paper presents an experimental study on concrete filled tubular stub columns. The environment friendly seawater and sea sand concrete (SWSS) and high corrosion resistance materials (i.e., stainless steel (SS) and carbon or basalt fibre reinforced polymer (CFRP or BFRP)) were adopted to make the stub columns. An appropriate mixture was firstly developed for SWSSC and the material properties of SS, CFRP and BFRP were determined by tensile coupon test and disk-split test. A total of 12 SWSSC filled columns and 12 corresponding hollow sectional tubes were tested under axial compression. The structural behaviour of test specimens was discussed. Finally, current design methods were used to estimate the ultimate capacity of SWSSC filled SS, CFRP and BFRP tubular stub columns.

KEYWORDS

SWSSC, stainless steel, CFRP, BFRP, concrete-filled stub columns.

INTRODUCTION

Concrete filled tubes, which are composed of core concrete and outer tubes, have been widely applied in civil engineering, such as bridges, high rise buildings, and power transmission towers. As the core concrete is confined by the encasing tubes, the strength of the concrete can be considerably enhanced by the so called “confinement effect”. Past researches (Zhao et al. 2010) have indicated that the confinement effect provided by circular outer tube is much higher than that provided by square or rectangular tubes.

It is well known that the huge consumption of Portland concrete causes environment problems (e.g. CO₂ emission due to the highly energy intensive production of Portland cement) and exacerbate the resource shortage (e.g. fresh water, river sand). An attractive solution is to replace the Portland cement, fresh water and river sand by industry waste (e.g. slag), seawater and sea sand respectively. Some studies (Mohammed et al. 2004) conducted on concrete made of seawater and (or) sea sand have found insignificant difference in the mechanical properties of seawater and sea sand concrete (SWSSC) and ordinary Portland concrete. However, the chloride ion introduced by seawater and sea sand can cause serious corrosion problems, which should be carefully considered before the application of SWSSC into engineering practice.

Because of its high corrosion resistance, the stainless steel (SS) is regarded as one of the desirable materials to replace the carbon steel used in marine structures. Several studies (Lam and Gardner 2008, Uy et al. 2011) have been conducted on concrete filled SS tubular columns. Their studies indicate that concrete filled SS tubes display more ductile behaviour than concrete filled carbon steel tubes and the current design methods for composite columns are conservative. A major reason for this different structural behaviour is that the material properties of SS, which has a rounded stress-strain curve, is different from carbon steel.

Even though many studies have been conducted on concrete confined by FRP wraps (Ozbakkaloglu 2013, Teng et al. 2009), the experimental investigations on concrete filled fibre reinforced polymer (FRP) tubes are rather limited. The FRP tube can provide strength in both longitudinal and hoop directions and can be used as formwork in new constructions. The reported researches mainly focused on concrete filled glass fibre reinforced polymer (GFRP) tubes (Fam and Rizkalla 2001, Zhang et al. 2015, Li et al. 2016). There is a need to study concrete filled
This paper presents an experimental investigation on seawater and sea sand concrete (SWSSC) filled stainless steel, CFRP and BFRP tubular stub columns. Both SWSSC filled circular tubes and corresponding hollow sectional tubes were tested. The material properties of SS and FRP were also obtained by tensile coupon test and disk-split test. Finally, the ultimate capacity of SS, CFRP and BFRP filled tubes is predicted using the existing design methods.

EXPERIMENTAL INVESTIGATION

Specimens

A total of 12 SWSSC filled tubular stub columns and 12 corresponding hollow section columns were tested in the present study. The specimens were made of SWSSC, SS, CFRP and/or BFRP. Four different nominal diameters (50, 101, 114, 165 mm) of outer tubes were adopted and the nominal thickness of the tubes was 3 mm. In order to eliminate the influence of global buckling and end effect, the specimen length is 150 mm for tubes with nominal diameter of 50 mm and 400 mm for all the other tubes.

The dimensions of test specimens are summarized in Table 1, where the ultimate capacity ($N_t$) is also given. The label of specimen consists of tube material type ("S" for stainless steel, "C" for CFRP, "B" for BFRP), nominal diameter ("50", "101", "114", "165"), and cross-section configuration ("H" for hollow sectional tubes, "C" for SWSSC-filled tubes).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$D_o$ (mm)</th>
<th>$t_o$ (mm)</th>
<th>$L$ (mm)</th>
<th>$N_t$ (kN)</th>
<th>Specimen</th>
<th>$D_o$ (mm)</th>
<th>$t_o$ (mm)</th>
<th>$L$ (mm)</th>
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<td>2.79</td>
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<td>B50-H</td>
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<td>0</td>
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<td>50.0</td>
<td>2.71</td>
<td>150</td>
<td>259</td>
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<tr>
<td>B101-H</td>
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<td>400</td>
<td>94</td>
<td>B101-C</td>
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<td>400</td>
<td>656</td>
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<tr>
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<td>124</td>
<td>B165-C</td>
<td>157.7</td>
<td>2.71</td>
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<td>1345</td>
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</table>

Material properties

The alkaline-activated slag-based concrete with seawater and sea sand was used in this research. A 1 m$^3$ SWSSC contained 360 kg slag, 190 kg seawater, 830 kg sea sand, 1130 kg coarse aggregate, 38.4 kg sodium meta-silicate, and 14.4 kg hydrated lime slurry. The 3% (percentage weight of slag) sodium meta-silicate activator was pre-blended with slag in the dry form before mixing. The hydrated lime slurry was used to improve the workability. More details of the SWSSC mixture can be found in Li et al. (2016).

Two batches of concrete were cast for specimens: batch 1 for SWSSC filled SS and CFRP tubes and batch 2 for SWSSC filled BFRP tubes. The concrete strength was obtained by standard cylinder test and the averaged 28-day strengths were 35.8 MPa for batch 1 and 32.8 MPa for batch 2. All the specimens and concrete cylinders were sealed with plastic film and stored at room temperature before testing.

The SS tubes are made of 316 grade austenitic stainless steel in accordance with AS/NZS4673 (2001). The material properties of stainless steel was obtained by tensile coupon test. Two coupons were cut from each size of tubes and the ends of coupons were flattened in order to be gripped by test machine. The tensile coupon test was
conducted in accordance with AS 1391 (2007) and the test results are summarized in Table 2, in which $f_{0.2}$ is 0.2% proof stress, $f_u$ is the ultimate stress for SS.

Both carbon fibre reinforced polymer (CFRP) and basalt fibre reinforced polymer (BFRP) were used in the present study. The FRP tubes were fabricated by the filament winding process with different fibre orientations. Based on the manufacturer data, 20%, 40%, and 40% fibres were in the angles of 15°, ±40° and ±75° with respect to longitudinal axis of tubes.

The longitudinal strength of FRP was obtained by the tensile coupon test. As the ends of coupons were not flat, two sets of grips made of aluminium were used to ensure that the test machine gripped the coupon tightly. The “disk-split”, similar to the test specified in ASTM D2290 (2012), was conducted on 20 mm wide FRP rings to evaluate the hoop strength. A detailed introduction of the test setup can be found in Li et al. (2016). Two coupons and two rings were tested for each size of FRP tubes and the averaged strengths in longitudinal and hoop directions are given in Table 2, in which $f_{ul}$ is longitudinal strength and $f_{uh}$ is hoop strength for FRP. As shown in Table 2, the hoop strength of FRP is much higher than the longitudinal strength and the strength of CFRP is higher than that of BFRP.

<table>
<thead>
<tr>
<th>Material type</th>
<th>$f_{0.2}$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>$f_{ul}$ (MPa)</th>
<th>$f_{uh}$ (MPa)</th>
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<td>562.1</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>S101</td>
<td>225.7</td>
<td>656.4</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>S114</td>
<td>280.7</td>
<td>617.8</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>S165</td>
<td>281.1</td>
<td>615.8</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>CFRP</td>
<td>N/A</td>
<td>N/A</td>
<td>235.8</td>
<td>592.8</td>
</tr>
<tr>
<td>BFRP</td>
<td>N/A</td>
<td>N/A</td>
<td>123.8</td>
<td>334.3</td>
</tr>
</tbody>
</table>

Test setup and instrumentation

For SWSSC filled tubes and SS hollow sections, the load was directly applied on the stub columns. End plates with annular slots were used for FRP hollow sections and the gap between the tube and slots was filled by thin steel strips (shims) in order to avoid the end failure. The loading rate for all the specimens is 1 mm/min with displacement control. For SWSSC filled specimens, cement paste was used to fill the gap caused by the shrinkage of concrete to make sure the load was simultaneously applied on the core concrete and outer tube.

Three linear variable displacement transducers (LDVTs) were equally placed around tested tubes and their averaged data were used to estimate the axial end shortening. Three longitudinal and three circumferential strain gauges were affixed to all columns at mid-height. All the loads, displacements and strains were automatically recorded by a data acquisition system. The test setup and instrumentation are similar to those described in Li et al. (2016).

Test results and discussions

The ultimate capacity ($N_u$), i.e., the maximum load the tube can resist, is presented in Table 1. The hollow stainless steel tubes failed by the plastic local buckling and yielding. The load-axial strain curves are shown in Figure 1, in which axial strain is the axial end shortening divided by specimen length. As shown in Figure 1, with the increase of diameter to thickness ratio ($D_o/t_o$), the axial strain corresponding to peak load and ductility decrease. Furthermore, the ultimate stress of the tube (i.e., ultimate capacity divided by cross-section area) to 0.2% proof stress ($f_{0.2}$) ratios are 1.90, 1.28, 1.21 and 1.14 for SS hollow sections with nominal diameter of 50 mm, 101 mm, 114 mm and 165 mm respectively.

The failure mode of CFRP and BFRP hollow tubes can be regarded as local buckling and crushing. As mentioned previously, the tube ends were restrained by end plates to avoid the end crushing. The sudden failure of all the hollow sections occurred near the end plates, which followed by a dramatic drop of the applied load. The load-axial strain curves are summarized in Figure 2, in which the post failure curves are not plotted. In general, the load-axial strain curves display a linear behaviour, and the ultimate axial strains for FRPs are much lower than those of SS hollow sections. It is also found that the hollow sections under compression cannot reach the longitudinal strength ($f_{ul}$) obtained in tensile coupon tests.
The failure mode of SWSSC filled SS tubes is an outward folding (Figure 3(a)) which is commonly observed for concrete filled steel tubes. The load-axial strain curves are plotted in Figure 4. A strain hardening response was observed for specimen S50-C due to the substantial confinement effect. For other specimens, there was no obvious load drop even though the specimens were under large deformation, which displayed great ductility. The test was terminated when the end shortening was close to the stroke of test machine, which was about 60 mm. The ultimate capacity \( N_t \) for SWSSC filled SS tubes is taken as the maximum load within the 5% axial strain and the \( N_t \) data are listed in Table 1. Similar definition of ultimate capacity is also adopted by Lam and Gardner (2008). A comparison between Figure 1 and Figure 4 indicates that the infilled concrete can delay the local buckling of SS tubes and the ductility is greatly improved.
The structural behaviour of SWSSC filled FRP tubes is different from that of SWSSC filled SS tubes. A bilinear response was observed for SWSSC filled FRP tubes (Figure 5). With the increase of applied load, FRP tubes firstly buckled in longitudinal direction with a slightly drop of applied load. However, the specimens can still sustain the load until the rupture of outer tube in hoop direction. Based on the test observation, the rupture of CFRP tubes was more sudden than that of BFRP tubes. For SWSSC filled CFRP tubes, after rupture, the applied load dropped to almost zero immediately (marked by “+” in Figure 5) and the inner concrete came out (Figure 3(b)). Nevertheless, the SWSSC filled BFRP tube still had residual strength after tube rupture and the failure is not as sudden as in the case of SWSSC filled CFRP tubes (Figure 3(c)).

As shown in Figures 4 and 5, the confinement effect is more obvious for SWSSC filled FRP tubes, but the SWSSC filled SS tubes have much higher ductility. The main reason is the different material properties of SS and FRP. The SS, with rounded stress-strain response, can provide passive confinement while the FRP, with linear stress-strain response, can provide active confinement.

PREDICTION OF ULTIMATE CAPACITY

SWSSC filled SS tubes

Currently, there are no design codes for concrete-filled stainless steel columns. A design method (DBJ13-51 2003) for concrete filled carbon steel tubes ($N_{DBJ}$) and the simple superposition method ($N_{sup}$) are adopted in the present study. The design formulas in DBJ13-51 are listed as Eqs 1 and 2:

$$N_{DBJ} = (1.14 + 1.02\xi_0) f_{cu} (A_s + A_c)$$ (1)

$$\xi_0 = A_s f_y / A_c f_{cu}$$ (2)

in which $\xi_0$ is confinement factor, $A_s$ is cross-section area of steel tube, $A_c$ is concrete area, $f_y$ is yield strength ($= f_{0.2}$ for SS), $f_{cu}$ is cubic concrete strength, which is 0.85$f'_c$. In the simple superposition method, the predicted capacity is the capacity summation of core concrete and outer tube.

A comparison between the test capacity and predicted capacity is shown in Table 3. It seems that both the simple superposition method and current design method are conservative. The main reasons are the confinement effect is ignored in simple superposition method and the strain hardening of SS is not considered in either of the two methods.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$f_{0.2}$ (MPa)</th>
<th>$f'_c$ (MPa)</th>
<th>$N_t$ (kN)</th>
<th>$N_{sup}$ (kN)</th>
<th>$N_{DBJ}$ (kN)</th>
<th>$N_{sup}/N_t$</th>
<th>$N_{DBJ}/N_t$</th>
</tr>
</thead>
<tbody>
<tr>
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<td>35.8</td>
<td>235</td>
<td>162</td>
<td>210</td>
<td>0.69</td>
<td>0.89</td>
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<tr>
<td>S101-C</td>
<td>225.7</td>
<td>35.8</td>
<td>570</td>
<td>457</td>
<td>506</td>
<td>0.80</td>
<td>0.89</td>
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<tr>
<td>S114-C</td>
<td>280.7</td>
<td>35.8</td>
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<td>663</td>
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<tr>
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<td>1207</td>
<td>1290</td>
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<td>0.01</td>
</tr>
</tbody>
</table>

SWSSC filled CFRP and BFRP tubes

Researchers (Ozbakkaloglu 2013, Lam and Teng 2003) have proposed some design methods for concrete confined by FRP wraps (fibre oriented in hoop direction only). However, there is no method for concrete filled FRP tubes that contain fibres in both hoop and longitudinal directions. The method proposed by Lam and Teng (2003) for concrete confined by FRP wraps is adopted herein to estimate the ultimate capacity of concrete filled FRP tubes ($N_{wrap}$). $N_{wrap}$ can be calculated by the following equations (the longitudinal strength is ignored):

$$N_{wrap} = A_c (f'_c + 3.3 f_l)$$ (3)

$$f_l = 2 f_{uho} / D_o$$ (4)

in which $A_c$ is concrete area, $f'_c$ is concrete strength, $f_l$ is confining stress, $f_{uho}$ is hoop strength of FRP, $D_o$ is tube diameter and $t_o$ is tube thickness. The predicted capacity by this method is listed in Table 4, in which the prediction by the simpler superposition method ($N_{sup}$) is also listed.

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As shown in Table 4, the simple superposition method is too conservative as the concrete strength can be significantly enhanced due to the confinement effect provided by FRP tubes. Lam and Teng’s method for concrete confined by FRP wraps is slightly conservative as the longitudinal strength of the FRP tube is ignored.

<table>
<thead>
<tr>
<th>Specimen</th>
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<th>$f_{cd}$ (MPa)</th>
<th>$f^c$ (MPa)</th>
<th>$N_l$ (kN)</th>
<th>$N_{sup} (kN)$</th>
<th>$N_{wrap} (kN)$</th>
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<th>$N_{wrap}/N_l$</th>
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<tr>
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CONCLUSIONS

This paper presents an experimental study on seawater sea sand concrete (SWSSC) filled SS, CFRP and BFRP tubular stub columns under axial compression. Several conclusions can be made from current study: (1) the concrete strength enhancement caused by SS tubes is lower than that by CFRP and BFRP tubes, but the ductility of SWSSC filled SS tubes is much higher; (2) Current design method for concrete filled carbon steel tubes can be conservatively applied to SWSSC filled SS tubes; (3) The design method for concrete confined by FRP wraps can be applied to SWSSC filled CFRP and BFRP tubes. There is a need to develop more accurate design models. Research is being conducted on the durability of SWSSC-filled SS tubes and FRP tubes.

ACKNOWLEDGEMENT

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REFERENCES

REDUCED ORDER APPROXIMATION OF THE LATTICE DISCRETE PARTICLE MODEL FOR THE SIMULATION OF FRP CONFINED CONCRETE COLUMNS

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ABSTRACT

It has been recently proved that the Lattice Discrete Particle Model (LDPM) is capable of accurately capturing the behavior of uniformly and non-uniformly confined concrete and, consequently, it can be considered a useful tool for the simulation of FRP confined concrete columns with different cross sections under compressive loading. The LDPM computational framework is implemented into a multi-purpose structural analysis program called MARS, which is based on an explicit dynamic algorithm scheme, advantageous in terms of convergence. However, decreasing the maximum stable time step with the highest natural frequency of the system, the computational time necessary to carry out simulations of quasi-static events, like the compression tests in the present study, might be highly demanding. In order to decrease the computational cost of the simulations, the Proper Orthogonal Decomposition (POD) as a model reduction technique has been explored for the application to the LDPM models of FRP confined columns and the relationship between efficiency gain and accuracy loss is discussed.

KEYWORDS

FRP, POD, LDPM, confinement, numerical model.

INTRODUCTION

Various numerical models have been explored in literature to reproduce the behavior of FRP-confined concrete columns and to capture the interaction between the axial strains and lateral expansion of concrete with the corresponding stress increase in the external jacket (e.g. Karabinis et al. 2008, Yu et al. 2010, Teng et al. 2015). Most of the macroscopic models available, though, have limitations related to the simplification in the constitutive laws for concrete softening response, so that the good agreement between experimental data and numerical models is often oriented to specific applications rather than generally valid (Wu, 2015). The mechanisms activated during the fracturing process of concrete are complex and not easy to be simulated with a simplified macroscopic mode: precise and accurate material dilatation properties during the micro cracking evolution and fracture propagations are crucial for capturing the post-peak response of FRP-confined concrete (Lam and Teng, 2003). For these reasons, a recently developed mesoscale model for concrete (Cusatis et al. 2011), called the Lattice Discrete Particle Model (LDPM) has been explored for simulations of FRP-confined response of concrete columns. After an improvement in the constitutive law in compression and the proper parameters calibration (Ceccato et al. 2015), the model can predict the behavior of FRP confined concrete and its sensitivity to the stiffness of the FRP wrapping and to the shape of the cross section.

The LDPM computational framework is implemented into the MARS code (Pelessone, 2015), which is a multi-purpose structural analysis program based on an object-oriented architecture. MARS performs structural analysis by an explicit dynamic algorithm based on a central difference scheme and is very effective in the management of the various computational entities (nodes, finite elements, loads, etc.) making possible the numerical simulation of very large systems. In addition, the explicit character of the computational scheme implemented in MARS makes it advantageous because is not affected by the convergence problems that implicit schemes often have in handling softening behavior. Explicit algorithms, however, are stable under conditions and require an accurate evaluation of the numerical stability of the numerical simulations. Decreasing the maximum time step with the highest natural frequency of the computational system, a prohibitive number of increments may be required for problems governed by low frequencies and, consequently, application of explicit dynamics to quasi-static events,
like the ones related to the present study, may become critical in terms of time analysis. The computational time necessary to carry out the simulations performed, in fact, turned out to be significantly demanding. Though, time steps much larger than the critical one would be sufficient to accurately describe the low-dynamic overall response and, in literature, different solutions have been investigated to artificially increase the speed of the process in simulations (e.g. Cocchetti et al. 2013, Gao et al. 2014, Chang, 2014). An interesting and promising approach can be found in the Proper Orthogonal Decomposition as a Model Reduction technique, which has now applications in different fields of engineering and physics (e.g. Behzad et al. 2014, Chen et al. 2014, Li et al. 2013). Dynamic systems can be projected onto subspaces containing the solution of the problem or a good approximation, so that a high-dimensional process is converted into a low-dimensional description of it.

**EXPLICIT INTEGRATION OF THE DYNAMICS EQUATIONS OF DISCRETE SYSTEMS**

**Full-Order Integration Algorithm for LDPM**

In this section, the algorithmic framework for the solution of the dynamics equation is presented for LDPM, and more generally for any discrete dynamic equilibrium problem solved through explicit integrators. The application of a reduced order method to this algorithm is discussed afterwards.

Let consider the motion governing equation in \( \mathbb{R}^n \), a domain which is discretized in space:

\[
\begin{align*}
\ddot{u} + (G \dot{u}) + f_{\text{int}}(u) &= f_{\text{ext}} \\
u(t_0) &= u^*, \quad \dot{u}(t_0) &= \dot{u}^*
\end{align*}
\]

(1)

defined \( \forall t \in [t_0, t_f] \) and where \( M \) is the mass matrix, \( u \) is the nodal displacement vector, \( f_{\text{int}} \) is the internal forces vector, \( f_{\text{ext}} \) is the external forces vector; \( u^* \) and \( \dot{u}^* \) denote the initial displacements and velocities. Neumann Boundary Conditions are applied in the space domain in the form \( \dot{u}_i = v_0 \forall X_i \in \mathcal{B} \subset \mathbb{R}^n \).

The system (1) can be discretized in time to be solved numerically, using the middle point rule as a time integrator (explicit algorithm):

\[
\begin{align*}
\ddot{u}^m &= M^{-1}(f_{\text{ext}}^m - f_{\text{int}}(u^m)) \\
\dot{u}^{m+1/2} &= \dot{u}^{m-1/2} + \dot{u}\Delta t \\
u^{m+1} &= u^m + u^{m+1/2} \Delta t
\end{align*}
\]

(2)

where \( \Delta t \) is the time step and \( m \in [1, n_{\text{steps}}], n_{\text{steps}} \in \mathbb{N}^* \) the number of the current time step. In elastic regime, the time step \( \Delta t \) must be subjected, because of stability conditions, to the constraint \( \Delta t < \frac{2}{\omega_{\text{max}}} \) where \( \omega_{\text{max}} \) represents the highest natural frequency of the computational system. Belytschko et al. (2000) showed that \( \omega_{\text{max}} < \max(\omega_i) \), where \( \max(\omega_i) \) are the natural frequencies of each element belonging to the mesh, so the stable time step can be computed solving the eigenvalue problem given by \( \det(K - \omega^2M) = 0 \) where \( K \) is the stiffness matrix and \( M \) is the mass matrix.

**Reduced-Order Integration Algorithm for LDPM through the Proper Orthogonal Decomposition**

*A brief introduction to POD*

Thanks to model reduction techniques such as POD, dynamical systems can be projected onto subspaces containing the solution of the problem, or a good approximation of it, so that a low-dimensional process can describe a high-dimensional problem accurately enough (Liang et al. 2001). The approximate solution can be sought in a low-order subspace, reducing the computational costs and storage requirements but still reproducing the main characteristic dynamics.

In particular, the POD technique allows to find the optimal orthogonal projection of a vector space \( \mathcal{V} \subset \mathbb{R}^n \), where the vectors of interest (the solution) take their value, onto a subspace \( \mathcal{S} \subset \mathbb{R}^k \) of dimension \( k < n \), containing the best approximation possible of \( \mathcal{V} \) in \( \mathbb{R}^k \).

Assuming that \( u \in \mathcal{V} \subset \mathbb{R}^n \) is the vector solution of the problem and \( \{\phi_i\}_{i=1}^n \) is a set of arbitrary orthonormal basis vectors spanning \( \mathcal{V} \), \( u \) can be written as a linear combination of the basis vectors through the coefficients \( d_i \):

\[
u = \sum_{i=1}^n d_i \phi_i = \Phi d
\]

(3)

where \( d = (d_1, d_2, ..., d_n) \) and \( \Phi = [\phi_1, \phi_2, ..., \phi_n] \).

POD finds the set of basis vectors, called the proper orthogonal modes, that can span a vector subspace of order \( k \) containing the most accurate approximation of the vector \( u, \sum_{i=1}^k d_i \phi_i (k \leq n) \).
Considering a finite dimensional system of $n$ state variables of interest (e.g., displacement) and capturing, at $m$ instants of time, a set of $n$ simultaneous measurements of these $n$ state variables, data can be arranged in an $n \times m$ matrix $U$. Each element $U_{ij}$ is the measurement of the $i$-th state variable taken at the $j$-th time instant. The final result of the data collection is assumed to be the $n \times m$ matrix $U$:

$$U = \begin{pmatrix}
  u_1(t_1) & \cdots & u_1(t_m) \\
  \vdots & \ddots & \vdots \\
  u_n(t_1) & \cdots & u_n(t_m)
\end{pmatrix} \tag{4}
$$

The Schmidt-Eckart-Young-Mirsky Theorem (Antoulas, 2005) shows the following statement:

- Let define the $n$-by-$n$ real symmetric matrix $C = UU^T$;
- Let denote by $\lambda_i$ the eigenvalues of $C$ and $\phi_i \in \mathbb{R}^n$, $i = 1, \ldots, n$ their associated eigenvectors such that $C\phi_i = \lambda_i\phi_i$, $i = 1, \ldots, n$;
- $\lambda_1 \geq \lambda_2 \geq \cdots \geq \lambda_n \geq 0$

The subspace optimizing the orthogonal projection of fixed rank $k$ is the invariant subspace of $C$ associated with the eigenvalues $\lambda_1 \cdots \lambda_k$. The corresponding eigenvectors $\phi_1 \cdots \phi_k$, spanning the subspace, are called proper orthogonal modes and can be collected into a matrix $B_k = [\phi_1 \cdots \phi_k]$ such that $B_k^T B_k = I$. This subspace is evaluated so that it is able to capture the majority of the variation in the model solution, by sampling the variable of interest from the full model for each node at different time. These samples are called snapshots and, if they are representative enough, they can be used in an interpolation scheme to approximate the full model. Being the displacement the variable of interest, the following equivalence holds:

$$u \approx \hat{u}(k) = \sum_{i=1}^{k} d_i \phi_i = B_k d; \tag{5}
$$

**Reduced Order Algorithm: integration scheme and boundary conditions**

The equation of motion (1) can be projected onto $\mathbb{R}^k$ through $B_k$:

$$B_k^T \mathbf{M} B_k d + (B_k^T \mathbf{G} B_k d) + B_k^T f_{\text{int}} = B_k^T f_{\text{ext}} \tag{6}
$$

and integrated numerically with the middle point rule:

$$\begin{cases}
  \bar{d}^m = (B_k^T \mathbf{M} B_k)^{-1} (f_{\text{ext}}^m - f_{\text{int}}) \\
  d^{m+1/2} = d^{m-1/2} + \Delta t_r \bar{d}^m \\
  d^{m+1} = d^{m} + \Delta t_r \bar{d}^{m+1/2}
\end{cases} \tag{7}
$$

The unknowns are the coefficients $d_i$, $i = 1, k$, guaranteeing that the linear combination of $B_k$ columns is as close as possible to the real solution. In each time step, the full dimensional displacement $\tilde{u}$ and the corresponding velocity $\tilde{u}$ are evaluated, from the associated projected values, because the internal forces and the residual forces are computed in the full original domain. The main reason for this choice is to keep the reduced system algorithm as close as possible to the original one, preserving all the explicit algorithm features, necessary when dealing with highly non linear constitutive laws, and allowing an easy transition from one to the other in the snapshot collecting process. The new time step $\Delta t_r$ must be subjected to a different constrain, $\Delta t_r < t_{cr} = 2/\omega_k \text{max}$, leading to the following eigenvalue problem: $\text{det}(B_k^T \mathbf{K} B_k - \omega_k^2 B_k^T \mathbf{M} B_k) = 0$.

The Proper Orthogonal Modes, through which the reduced system is computed, will automatically satisfy any fixed boundary conditions in terms of displacements (or velocities) for construction. In fact, the matrix $U$ has a 'zero' row corresponding to any fixed degrees of freedom and this information will be transferred to the POD MODEs themselves and, consequently, to any object in the subspace they span. The non-zero BCs need to be re-applied in the reduced system, these data not being preserved by the modes. It is not easy however, to apply the BCs directly to the projected degrees of freedom in the reduced subspace, because this would lead to an overdetermined system. To elude the obstacle, the BCs are applied indirectly as equivalent external forces through the penalty method: $f_{\text{ext}} = K_p (u_p - u)$, where $f_{\text{ext}}$ is the penalty force equivalent to the BC, set around $10^3$ times the element stiffness, $u_p$ the penalty displacement, whose value comes from the BC, and $u$ is the current displacement of the node of interest. Due to instability issues, the node mass $M_i$ where the penalty constrain is applied is artificially increased (mass scaling) by adding a quantity which is proportional to the penalty coefficient $K_p$ and to the square time step $\Delta t$: $M_{ii} = M_{ii} + 1.1 \cdot K_p \cdot \Delta t^2$. 279
Snapshot collection

The snapshots can be computed \textit{a posteriori}, after the end of the regular analysis. This method allows the definition of the reduced system for model validation purposes and the snapshot collection can be the most efficient possible, since the real solution is already known. Aiming to a predictive tool, though, the snapshots should be collected during the analysis itself, \textit{in itinere}, alternating the integration in the complete system, for the snapshots collection, with the integration in the reduced system, until the snapshots previously collected are enough representative of the response. When dealing with quasi static problems, the complete system can be integrated only for a small initial time interval $\Delta T$, to capture an adequate number of snapshots, which sufficiently describe the system behavior. Then, the corresponding reduced system can be computed from the first $k$ modes and the analysis carried out in the reduced system with a greater time step. The snapshots may need to be updated to take account of any variation of the external (for instance in case of changes in applied forces, displacements, velocities) or internal conditions (for instance changes due to the constitutive law). When, after a time interval $\Delta T_R$, the old snapshots are not representative anymore of the response, the analysis should continue using the full order system, to collect a new group of snapshots and compute a new subspace. The amplitude of $\Delta T_R$ can be fixed and different ratios of them have been related to the accuracy of results. Once the first snapshots have been collected and the corresponding reduced space defined, the integration in the reduced system can start, using as initial condition for the integration the following values: $\ddot{u} = B_k^T \dot{u}$; $\dot{u} = B_k^T \ddot{u}$, where $\ddot{u}$ and $\dot{u}$ are the values of velocity and displacement given by the last time step integration in the complete system.

POD APPLICATION TO THE NUMERICAL STUDY OF FRP-CONFINED COLUMNS

Preliminary Studies

The new algorithm based on the POD method has been preliminarily tested for 1D truss elements and 2D beam elements, finding out that the efficiency of the method, based mainly on the gain in terms of stable time step, increases with the number of degrees of freedom of the original problem.

Also, just a very limited number of modes are enough to achieve high levels of accuracy of the solution: only one in case of linear response, two in case of softening. If yielding or fracture occurs at the material level, in fact, the subspace must be redefined. The POD modes can be interpreted as shape functions with a global support, as shown in Figure 2, where the dynamic oscillations are filtered by taking into account only the main modes. The gain in terms of CPU time ($T_F$: CPU time for the full order $T_R$: CPU time for the reduced order algorithm) is increasing from $PIF = T_F/T_R \equiv 7$ for 25 elements to $PIF \equiv 110$ for 200 elements in the linear response and from $PIF = T_F/T_R \equiv 4$ for 25 elements to $PIF \equiv 60$ for 200 elements in the softening response. The same algorithm was then tested for 3D simple specimens modeled with LDPM.
FRP-confined concrete with LDPM and POD

To pursue the final aim of this research, the POD algorithm has been applied to columns subjected to compressive loadings and confined with FRP sheets (see Figure 3a, $H = 300$ mm, $D = 150$ mm). The geometry and material parameters are accurately described in Ceccato et al. 2015; in this application, the FRP jacket and the external surface of the concrete column are sharing the same nodes. Figure 3b shows the numerical results from the reduced order algorithm compared with the ones from the full algorithm and the experimental data. Similarly to the 1D and 2D tests, the linear response is easily captured by the reduced algorithm with the 1st mode only, while the post-peak response needs at least the 1st and 2nd mode. However, frequent updates (which are automatically set) are required after the elastic phase, because of the progressive and diffused fracturing process of the material, which makes the POD modes almost immediately no longer representative of the ongoing response. In this specific case, the stable time step in the reduced algorithm is 60 times greater than the full algorithm, but the index of performance is limited to $\text{PIF} = 3$. In fact, the advantage in terms of time step is partially lost because of the frequent updates. The algorithm has still room for improvement and the objective would be to achieve a global gain of 10 times so that LDPM can be used also for practical applications. Anyway, the algorithm as it is can still be considered useful in the calibration process, to check the trend of the response in a shorter time. The oscillations in the reduced order curve can be minimized with a smoothing algorithm.

CONCLUSIONS

A three-dimensional mesoscale model based on the Lattice Discrete Particle Model (LDPM) had been successfully developed for the simulation of FRP confined concrete columns subjected to compressive loading. To cope with
the high computational costs of the numerical simulations of LDPM problems, the Proper Orthogonal decomposition (POD) has been explored as a model reduction technique, in order to speed up the analysis. After some preliminary studies, the method has been applied to the case of interest: the response of FRP confined circular columns has been evaluated, comparing the numerical results with the fully explicit algorithm to the ones with the reduced algorithm. The reduced order response approximates the regular LDPM simulation, with some oscillations that can be minimized with a smoothing algorithm. Even if the gain in terms of stable time step is consistently high, POD modes need to be updated quite frequently during the inelastic phase in compression so, the global gain in terms of CPU time remains limited. The reduced solution can still be considered a good approximation and be a useful tool in the LDPM calibration process, and the authors are still working on improvements of the POD application for LDPM in compression, aiming to achieve a higher global CPU time gain.

REFERENCES

EFFECT OF EXTERNAL CONFINEMENT ON PLASTIC HINGE OF RC COLUMNS

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ABSTRACT

The present paper intends to investigate the behaviour of plastic hinge of fibre reinforced polymer (FRP)-confined RC columns numerically using three-dimensional finite element method (FEM). As FEM results largely depend on the model and its parameters, the FEM model is first calibrated with test results to show that the results are reliable even different models/parameters are adopted. To gain insights into the influence of FRP confinement ratio, lengths in the plastic hinge zones involving rebar yielding zone, concrete crushing zone, and curvature localization zone are investigated in detail. The FEM results show that all three zones are significantly affected by FRP confinement. Both the lengths of the rebar yielding zone $L_{sy}$ and the curvature localization zone $L_{pc}$ increase first and then decrease as confinement ratio increases, while the length of concrete crushing zone $L_{cs}$ keeps decreasing with the increase in the confinement ratio.

KEYWORDS

Plastic hinge, FRP, RC columns, confinement, FEM.

INTRODUCTION

In recent years, fiber-reinforced polymer (FRP) jackets have become popular in providing confinement to reinforced concrete (RC) columns in construction industry (Ibell et al. 2009). A sufficient deformation for RC columns can be achieved by FRP confinement at a potential plastic hinge region (Seible et al. 1997; Saadatmanesh et al. 1997). The plastic hinge region is defined as the physical region over which the member experiences inelastic deformations and severe damage. Qualification of plastic hinge length $L_p$ for FRP-confined columns is critical not only for the design of new members but also for the rehabilitation of load structures.

Due to high nonlinearity of materials and complicate interactions between constitutive materials involved, the behavior of plastic hinge of FRP-confined RC members has been previously focused on experimental study. Priestley et al. (1996) and Elsanadedy and Haroun (2005) showed that $L_p$ of an FRP-confined RC column is smaller than that of a traditional RC column. However, some experimental results have shown that $L_p$ of FRP-confined columns is larger than that of traditional RC columns (Ozbakkaloglu and Saatcioglu 2006). Other researchers reported that the $L_p$ of FRP-confined columns is equal to that of a traditional RC column (Monti et al. 2001; Binici 2008). Jiang et al. (2014) observed that $L_p$ first increases and then decreases with FRP confinement ratio. Hence, there is no consensus among researchers on the qualification of $L_p$. The literature reveals contradictory conclusions about the plastic hinge length of FRP-confined RC columns. Based on these findings, it is clear that $L_p$ of FRP-confined RC columns needs further investigations.

Previous studies on finite element modeling of FRP-confined RC members have focused on the strength and ductility of structural members (Kachlakiev et al. 2001; Mostofinejad and Talaeitaba 2006; Ibrahim and Mahmood 2009; Abdelkarim and ElGawady 2014) and few concerned the plastic hinge modeling. This paper aims to investigate the plastic hinge problem of FRP-confined RC columns in greater detail through finite element method (FEM). The FEM model is first calibrated with test results. After that, a systematic parametric study was subsequently carried out to investigate the effect of the confinement ratios on the rebar yielding zone, concrete crushing zone and curvature localization zone of FRP confined RC columns.
FINITE ELEMENT MODELLING AND IMPLEMENTATION

In the present work, the plastic hinge regions of FRP-confined RC columns are investigated in a detailed manner by numerical simulation. General finite element software ABAQUS (2010) is used in this work. The constitutive models involve concrete, steel reinforcement, FRP and interfaces between concrete-steel reinforcement and concrete-FRP. Detailed modeling and the parameters are summarized below.

The 8-node linear brick element C3D8R in ABAQUS is employed for concrete elements. The mesh size of the concrete element for RC columns is 20 mm. Elements smaller than 20 mm make little difference to the results, according to the mesh convergence study.

Damaged plasticity model is adopted to model the material behavior of concrete. The ascending section of the tensile relationship is assumed to be linear and softening is assumed to be exponential. The fracture energy method is adopted for post-peak cracking of concrete. The strain at peak stress \( \varepsilon_{cr}^c \) is assumed to be 0.0001. The fracture energy for concrete \( G^f \) is assumed to be 170 N/m (Hillerborg 1985).

In compression, the analysis-oriented stress-strain model by Teng et al. (2007) is employed for both unconfined and confined concrete. The default values in ABAQUS are adopted for yield criterion parameters of unconfined concrete. Since the flow rule for confined concrete is significantly different from the unconfined, variations of dilation angle for the two cases are very different (Yu et al. 2010; Jiang and Wu 2012). The dilation angle of 30° is selected for unconfined concrete and the yield criterion parameters and the flow rule parameters proposed by Yu et al. (2010) is employed for confined concrete.

A 2-node truss element T3D2 in ABAQUS is employed for steel reinforcement. The bilinear model which considers strain hardening is adopted to model the stress-strain relationship of the steel reinforcement.

The Spring2 element is selected to define the nonlinear behavior between concrete and longitudinal tensile reinforcement. The concrete nodes and steel nodes with the same coordinates are connected by springs. A large linear stiffness is applied in the normal direction of the interface, while the unified bond-slip relationship proposed in (Wu and Zhao 2012) is applied in the tangential direction to consider the nonlinear slip between concrete and steel bars. The bond stress-slip relationship can be transformed into the nonlinear spring force-relative displacement relationship.

The FRP jacket is modeled by 4-node membrane elements M3D4R and is treated as linear elastic laminar with orthotropic elasticity. Elastic modulus of FRP jacket is assigned only in the fiber direction, which means that the FRP jacket has stiffness only in hoop direction and provides confinement effect to concrete. The FRP sheet fails when its rupture strain is reached. Perfect bond between FRP sheet and concrete is assumed.

MODEL VERIFICATION

![Displacement vs Moment](image1)

![Curvature vs Moment](image2)

Figure 1 Load versus deformation curves

Two FRP confined RC columns (one square and another circular), are simulated herein using the above described FEM model. Specimens RS-1 (Ozbakkaloglu and Saatcioglu 2004) and ST-4NT (Sheikh and Yau 2002) were tested under constant axial loading and cyclic lateral deformation. It can be seen from Figure 1 that the initial stiffness and load-carrying capacity from the numerical results match well with those of measured results. Reasonable agreements are observed between measured load-deformation responses and FEM results.
PLASTIC HINGE ANALYSIS OF FRP-CONFINED RC COLUMNS

The influence of FRP confinement on the development of plastic hinge region is analyzed in detail for FRP-confined RC columns in this section. Nine specimens with FRP confinement ratios ranging from 0 to 0.466 (corresponding to 0 to 2 layers of FRP jacket) are studied. The confinement ratio \( \lambda_f \) is defined as follows:

\[
\lambda_f = \frac{f_{f}}{f_{c}} = \frac{2E_f \varepsilon_{f \mu} f_{f}}{bf_{c}}
\]

where \( f_{f} = 30 \text{ MPa} \) is the compressive strength of unconfined concrete; FRP fracture strain \( \varepsilon_{f \mu} = 0.0171 \); elastic modulus \( E_f = 245 \text{ GPa} \); and thickness \( t_f = 0.167 \text{ mm} \). The lateral displacement is monotonically increased under a constant axial force. The axial force is fixed to be 0.2\(N\), where \( N = f_{s} A_{s} \) in which \( A_{s} \) is the gross cross-sectional area of the column. All specimens have the same geometric dimensions and material properties. Due to the symmetry of the column, half of the column is modeled. The specimen sizes are 200 x 200 x 800 mm connected to a 300 x 400 x 900 mm stub (Figure 2). The corner radius of the columns is 20 mm and the effective cantilever length is 730 mm from the center of the loading to column base. For all specimens, steel bars of 16 mm diameter are employed as the longitudinal bar and steel bars of 8 mm diameter and spacing of 100 mm are adopted as transverse reinforcement. The steel has a yield strength of \( f_{s} = 400 \text{ MPa} \), an ultimate strength of \( f_{u} = 579 \text{ MPa} \) and elastic modulus of \( E_s = 194 \text{ GPa} \). The reinforcement details and geometric dimensions of the columns are as shown in Figure 2.

Investigation of rebar yielding zone

The regions where the strain of tensile reinforcement has reached or exceeded its yielding value is defined as the rebar yielding zone. Figure 3(a) shows the development of rebar yielding zone of FRP-confined RC columns with different confinement ratios. It can be found that the length of rebar yielding zone increases significantly with the increase of the column deformation after it yields and keeps almost constant after the drift ratio of 8%, which means that the maximum length of rebar yielding zone \( L_{sy} \) is limited to a certain value. This observation is consistent with the test results in literature (Scott 1996).

The influence of FRP confinement ratio on \( L_{sy} \) and moment capacity at the drift ratio of 8% is shown in Figure 3(b). \( L_{sy} \) first increases and then decreases as the confinement ratio increases. However, the maximum moment value of the columns always increases with increase of the confinement ratio. The compressive strength of confined concrete increases with the confinement ratio, resulting in an increase in the lever arm of the moment and hence an increase in the cross-sectional moment capacity. In the meantime, the yield moment does not change and hence the yielding section moves up along the column with the increase in the moment capacity at the column base. As a result, the distance from the column base to the first yielding section increases, which increases \( L_{sy} \). However, the frictional bond between longitudinal tensile reinforcement and concrete increases linearly with increase in the confinement ratio, which opposes the rebar stress and has an adverse effect on the strain penetration length and hence \( L_{sy} \). In other words, both moment capacity and bond stress increase with the increase in the confinement ratio. On the other hand, \( L_{sy} \) is dominated by moment increase when the confinement ratio is lower than 0.2. After that, the growth rate of moment capacity slows down when confinement is high and then the other mechanism,
friction, dominates the variation of $L_{sy}$. As a result, $L_{sy}$ first increases and then decreases with the increase in FRP thickness. This conclusion is consistent with Gu et al. (2010) and Jiang et al. (2012).

Investigation of concrete crushing zone

To study the damage zone of FRP-confined RC column, the lengths of compression region $L_{cs}$ and $L_{cc}$, are investigated, where $L_{cs}$ is defined as the length of the region where the concrete compressive strain is larger than the strain at peak stress of unconfined concrete (0.002) and $L_{cc}$ is defined as the length of the region where the compressive strain is greater than the crushing strain of unconfined concrete (0.006). It is found from the simulation results that the stable region of $L_{cs}$ moves towards the column base as the confinement ratio increases, which means that the damage zone of concrete becomes smaller when confinement increases, and concrete strain with a lower confinement distributes more evenly than that with a higher confinement. The compressive strain contours with different confinement ratios at the drift ratio of 8% are shown in Figure 4. The regions with strain value greater than 0.002 and 0.006 are displayed in the figure. It can be found that the area and height of the display regions decrease with the increase of confinement ratio, which means that confinement reduces $L_{cs}$ and $L_{cc}$. This is consistent with the experimental observations that higher level of FRP confinement results in a smaller damaged zone (Iacobucci et al. 2003; Ozbakkaloglu and Saatcioglu 2006). It can be found from the simulation results that there are sudden increases of strain value when it exceeds 0.002, which corresponds to the inflection point of axial stress-strain curve of unconfined concrete, after which the unstable process of crushing of concrete begins. With the increase in FRP confinement, the compressive strength of concrete increases and the area of compression failure reduces, leading to a smaller $L_{cc}$. 

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**Figure 3** Effect of FRP confinement ratio on rebar yielding zones

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**Figure 4** Concrete compressive strain contour at drift ratio of 8%
Investigation of curvature localization zone

Figure 5 Curvature distribution of FRP-confined RC columns

Figure 5 shows the curvature distributions at different deflections beyond yielding for FRP-confined RC columns. It can be seen that the curvature varies significantly along the column length due to concrete cracking. After a column yields, most of the plastic curvature gets concentrated within a certain zone. A dividing point exists from which to the column base the curvatures increase rapidly after yielding while those outside the region remain almost constant. This numerical result is consistent with experimental observations reported in literature (Hines et al. 2004; Gu et al. 2010). The distance from the dividing point to column base is defined as the length of significant curvature localization zone \( L_{pc} \) in this work.

The influence of confinement ratio on \( L_{pc} \) can also be found in Figure 5. It is clearly seen that the dividing point mentioned above moves toward the column base when the confinement ratio increases. However, the maximum curvature value increases from \( 3.83 \times 10^{-4} \) mm\(^{-1} \) to \( 4.36 \times 10^{-4} \) mm\(^{-1} \) when FRP layers increase from 0.5 to 2. Both tensile strain and compressive strain contribute to curvature. Therefore, curvature concentration is caused not only by rebar yielding but also by concrete plastic deformation.

CONCLUSIONS

This paper investigates the plastic hinge region of FRP-confined RC columns by the finite element method. The accuracy of the FEM model was first calibrated with test results. A systematic parametric study was subsequently carried out to investigate the effect of the major parameters on the rebar yielding zone, concrete crushing zone and curvature localization zone of FRP confined RC columns. The plastic hinge length of FRP-confined RC columns is very different from that of normal RC columns. Lengths of both the rebar yielding zone \( L_{sy} \) and the curvature localization zone \( L_{pc} \) increase first and then decrease as the confinement ratio increases, while the length of concrete crushing zone \( L_{cs} \) keeps decreasing with the increase in the confinement ratio.
REFERENCES


Strengthening of Structures with FRP Composites
COMPARISON OF MODELS FOR PREDICTING THE BEHAVIOURS OF FRP STRENGTHENED CONCRETE BEAMS CONSIDERING COMPRESSIVE MEMBRANE ACTION

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ABSTRACT

Recent research has shown that compressive membrane action (CMA) has a significant benefit in enhancing the load bearing capacities of FRP strengthened concrete beams. Generally two methods are used to predict the behaviour of FRP strengthened concrete beams with consideration of CMA. One method is using plasticity theory to obtain the beam’s resistance under CMA by considering strain compatibility and force equilibrium at the sectional level. Another main method accounts for the ultimate load of a laterally restrained beam by taking the sum of the bending capacity and the additional three-hinge arch load due to CMA. These two methods are compared analytically in this paper from the perspective of load bearing capacity. Results show that, when compared with the second model, the first model is more accurate to predict the load bearing capacity of a FRP strengthened concrete beam considering CMA because the first method considers CMA to be initially associated with the bending capacity.

KEYWORDS

Compressive membrane action, FRP strengthening, RC beams, model comparison.

INTRODUCTION

Since compressive membrane action (CMA) was first recognized by Turner in 1909 (Turner 1909), much work has been done on this topic particularly in the field of reinforced concrete (RC) slabs, as can be seen in the detailed overview by Taylor (2002). Research results have shown that CMA is beneficial for strength enhancement in normal concrete structures and FRP strengthened concrete structures (e.g. Taylor 2003; Taylor and Mullin 2006; Muthu et al. 2007; Zheng et al. 2007; Wang et al. 2011; Valipour et al. 2013; Zeng et al. 2015a, 2015b). With regard to the investigation of CMA (i.e. the prediction of load bearing capacity) in concrete beams, on the one hand a commonly applied method first proposed by Park (1969), is using the plastic theory to obtain the beam’s resistance. This method (Method I) considers CMA to be initially associated with the bending capacity and is good at estimating the CMA capacity of laterally restrained concrete beams. On the other hand, another method proposed by Rankin and Long (1997) is also widely adopted. This method (Method II) evaluates the load bearing capacity under CMA as the sum of the load bearing capacity due to bending capacity and the additional capacity due to the CMA effect. Both methods are proven to be effective in predicting the load bearing capacity of the laterally restrained concrete beams and a basic comparison can be found in review publications (e.g. Taylor 2002). However, the application of these two methods is not clear in the context of FRP reinforcements. This paper presents an analytical comparison between these two methods regarding the prediction of load bearing capacities of FRP strengthened concrete structures.

METHOD OF SOLUTION

General Assumptions

The following basic assumptions apply for both methods:
1) Plane sections remain plain.
2) The considered one-way concrete beam is completely symmetrical regarding geometry, reinforcement, loading and deformations.
3) The major cracks at the central and end sections after hinge formations (yielding) divide the beam into three rigid portions which keep flat until the ultimate state, as shown in Figure 1a. The length ratio of the left portion to the whole beam is assumed as $\beta$. The state of the left portion after yielding is shown in Figure 1b and 1c for the two methods to be mentioned below. 

4) Concrete stresses are characterized by an idealized equivalent rectangular block with an ultimate strain of $3.5\%$ (fib Bullein 14 2001); 

5) Steel reinforcement is believed to be perfectly elastoplastic and FRP is viewed to be linear and intact (no debonding or fracture occurs) with an ultimate strain of 1.5% if not specified.

**Method I**

In this method, plasticity theory is made use of to obtain the beam’s resistance under CMA by considering strain compatibility and force equilibrium at the sectional level (Park 1969). Recently, this method was extended to the application for FRP strengthened concrete beams (Zeng et al. 2015a). The method is briefly explained below. For detailed information, please refer to Zeng et al. (2015a). In this method, the axial force at beam end ($N_1$) is assumed to be equal to that at the mid-span ($N_0$) i.e. $N = N_0 = N_1$, and a constant elastic compression strain distribution along the span is assumed. With the assumption above, the compatibility requirement from Figure 1a can be expressed as

$$\frac{\beta l + t + 0.5\varepsilon (1 - 2\beta)l}{\cos \theta} = \frac{(1 - \varepsilon)\beta l}{2} + \left(\frac{h}{2} - x_0\right) \tan \theta + \left(\frac{h}{2} - x_1\right) \tan \theta$$

(1)

where $x_0$ and $x_1$ are the neutral-axis depths at the span and at the beam ends, respectively; $h$ is the depth and $l$ is the length of the beam; $\theta$ is the rotation angle; $t$ is the lateral displacement of the surrounding restraints and $\varepsilon$ is the uniformly distributed strain along the considered portion. By implementing $t = N/K$, $\varepsilon = N/E_cA_c$ and $\tan \theta = \delta / \beta l$, Equation (1) is simplified to:

$$h - x_0 - x_1 = \frac{\delta}{2} + \frac{N \beta l^2}{2\delta} \left(1 + \frac{2}{E_cA_c} + \frac{2}{KL}\right)$$

(2)

in which $\delta$ is the center deflection; $E_c$ is the elastic modulus; $A_c$ is the concrete area of the cross-section; and $K$ is the axial stiffness of the surrounding lateral restraints.

For a model of double-reinforcement sections, the internal axial force equilibrium can be written as

$$N = C_{c0} + C_{c1} - T_0 - F_0 = C_{s0} + C_{s1} - T_1 - F_1$$

(3)

where $C_{c0}$ and $C_{c1}$ are the concrete compressive forces, $C_{s0}$ and $C_{s1}$ the steel compressive forces, $T_0$ and $T_1$ the steel tensile forces, and $F_0$ and $F_1$ the FRP tensile forces, acting on sections at the mid-span and the beam end, respectively.
Considering that all terms in Equation (3) can be expressed by given geometric and material properties and the unknowns $x_0$ and $x_1$ based on a sectional force equilibrium analysis, $x_0$ and $x_1$ are obtained by solving the Equations (2) and (3) simultaneously for a given $\delta$. Further, the moments at the mid-span and the beam ends can be calculated as well as the load bearing capacity. When repeating such procedures for different values of $\delta$, a series of the load bearing capacities are obtained, and the maximum value can be seen as the ultimate load bearing capacity. Consequently, the load bearing capacity for a pointed load pattern is determined by

$$P_I = \frac{2(M_{u0} + M_{u1} - N\delta)}{\beta l}$$

where $M_{u0}$ and $M_{u1}$ are the ultimate moments at the span and the beam ends, respectively.

**Method II**

As mentioned above, in the second method the load bearing capacity of a lateral restrained beam is considered as the sum of a bending capacity and an additional capacity. This idea is adopted in this paper and the formulae are given separately.

**Bending capacity**

The bending moments, $M_{b0}$ and $M_{b1}$ at respectively the beam mid-span and ends, can be easily obtained by moment-curvature analysis and the corresponding bending capacity regarding a pointed load pattern can be calculated by equalling the external work of vertical load, $P_b$, and the internal work of the resistant moments at yielding

$$P_b = \frac{2(M_{b0} + M_{b1})}{\beta l}$$

**Additional capacity**

With respect to calculating the additional capacity, the main task is to obtain the level arm, $a$, as shown in Figure 1b. For the internal compressive force and the compatibility requirement, Equations (2) and (3) still apply. Considering the shape of the compressive stress of concrete after yielding, the level arm is estimated by

$$a = h - \delta - \frac{1}{2}(x_0 + x_1)$$

Then the additional capacity, $P_a$ is given as

$$P_a = \frac{2Na}{\beta l}$$

The calculation procedure is almost the same as that in Method 1: Equations (2) and (3) are solved for a give initial deflection $\delta$. The outcome values of $x_0$ and $x_1$ are used to calculate the additional capacity in Equation (7). Then the calculation procedure is repeated with a larger deflection. The iteration stops when the obtained additional capacity is smaller than that in the previous step, which is taken as the ultimate additional load bearing capacity. Finally, the load bearing capacity under CMA consideration is the sum of bending capacity and the additional capacity, i.e.

$$P_{II} = P_b + P_a$$

**COMPARISON OF RESULTS AND DISCUSSION**

**Qualitative Comparison**

These two methods are first compared qualitatively. Based on the theories above a conclusion is directly drawn: these two methods share the same kinetical mechanisms. This is straightforward as they share the same main assumption that the formed hinges due to yielding divide the considered beam into three rigid portions which keep flat until the ultimate state. This assumption leads to the analysis of geometries after deformation and further to Equation (2). Furthermore, it is logical to believe that the compressive forces at the beam mid-span and ends are equal, which gives rise to Equation (3).
The difference between these two methods is also evident. The interaction effect between the bending moment and the axial force is combined in Method I, whereas in Method II this effect is not explicitly considered.

**Quantitative Comparison**

These two methods are compared quantitatively by a case study. Considering that in FRP strengthened concrete structures the load bearing capacity is at least enhanced by the FRP strengthening and the effect of CMA, it is necessary to select a dimensionless quantity to differentiate the enhancement due to FRP strengthening and the enhancement due to the CMA effect. Therefore, a strength enhancement factor to evaluate the effect of CMA and/or the effect of FRP strengthening is adopted and defined as \( \alpha_{p,FRP} = \frac{P_{FRP}}{P_0} \) and \( \alpha_{p,CMA,FRP} = \frac{P_{CMA,FRP}}{P_0} \), where \( \alpha_{p,FRP} \) and \( \alpha_{p,CMA,FRP} \) are the enhancement factors considering the enhancement of FRP, and of FRP and CMA together, respectively; \( P_{CMA,FRP} \) is the peak resistance considering CMA calculated by methods in this paper, \( P_0 \), and \( P_{FRP} \) are the beam peak resistances calculated by *fib* bulletin 14 (2001), each time for normal concrete beams and FRP strengthened concrete beams, respectively.

A beam similar to the four-point loaded two-span beam in Vasseur *et al.* (2006) is adopted as the benchmark beam in this parametric study, as sketched in Figure 2a. This 10 m beam with a 200 \( \times \) 400 mm section and a value of 0.2 for \( \beta \) is assumed to be laterally restrained (\( K = 10^8 \) kN/m) at both ends. C30/37 concrete is used and three steel bars are initially placed along the beam length at both the beam top (2\( \phi \)12+1\( \phi \)18) and the beam bottom (2\( \phi \)12+1\( \phi \)20). The elastic modulus and the yield stress of the steel bars are 200 GPa and 500 MPa, respectively. In cases of FRP strengthening, a CFRP layer with a thickness of 1.2 mm is applied along the beam at the location of tension zones and the elastic modulus and the ultimate strain of the FRP are considered 190 GPa and 1.5%, respectively. All these values apply if no further information is indicated.

By using methods I and II, Figure 2 shows the results of the enhancement factors for varying ratios of FRP reinforcement. Figure 2a clearly indicates that the load bearing capacity increases with increasing FRP reinforcement. However, the enhancement due to the CMA effect decreases if the beam is increasingly reinforced, as shown in Figure 2b.

In regard to the difference of the prediction of load bearing capacity using these two methods, it can be concluded from Figure 2a that Method II is generally more conservative than Method I, which also applies to the case of the enhancement due to the CMA effect as shown in Figure 2b. Figure 2 further also shows that the difference is negligible if the considered beam is heavily reinforced. For example, in this specific case study, the difference between these two methods is about 1% for the overall enhancement and 5% for the enhancement due to the CMA effect, as long as the FRP reinforcement ratio is larger than 0.12%.

These findings are consistent with the conclusions in Zeng *et al.* (2015a) and Wang *et al.* (2011). On the one hand, the predictions of Method I in Zeng *et al.* (2015) show approximately 5% overestimation compared to the test results. On the other hand, the predictions in Wang *et al.* (2011) using a method similar to Method II show about 15% underestimation compared to the test results. Therefore, from the perspective of accuracy, it is believed that Method I is relatively more accurate than Method II. This could be explained by the fact that Method I considers the effects of bending and compression to be inherently related, which is corresponding to reality. However,
because Method I overestimates the load bearing capacity while Method II underestimates the load bearing capacity, an average of the results from these two methods could be considered.

CONCLUSIONS

Two methods of prediction of the load bearing capacity of laterally restrained FRP strengthened concrete beams are summarized in this paper. These two methods are compared qualitatively and quantitatively using a case study. Results show that in general Method II is more conservative than Method I. Together with the findings of previous research a combination of these two methods i.e. an average of the predictions of these two methods could be considered.

ACKNOWLEDGEMENT

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REFERENCES

FORCE DISTRIBUTION BETWEEN INTERIOR STEEL REINFORCEMENT AND EXTERNALLY BONDED CFRP OF RC BEAMS UNDER CYCLIC LOADING

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ABSTRACT

The distribution of forces between interior steel and externally bonded reinforcement (EBR) of reinforced concrete members is normally determined assuming a plane strain distribution. But the different bond characteristics of the reinforcement types lead to a force distribution, which can be different from the determined distribution of forces. Experimental test results show that the strain in the EBR is higher than assumed particularly at low strain values, as they occur in the load cases of fatigue. In case of cyclic loading the difference between assumed and measured strain is reduced as a result of bond damage. However, an accurate prediction of the force distribution is only possible if the bond behavior of EBR and the cast-in reinforcement under cyclic loading is considered.

In this paper a model for the determination of the force distribution between internal and external reinforcement under fatigue loading is presented. For the validation of the model combined shear and pull out tests under cyclic loading were carried out. The test results can be recalculated using a formulation for the redistribution of internal forces under cyclic loads considering the bond behavior under cyclic loading for both reinforcement types.

KEYWORDS

CFRP, RC beams, strengthening, interfacial stresses, analytical solution.

INTRODUCTION

Many reinforced concrete bridges in Germany show damages caused by aging and overloading. Strengthening with externally bonded carbon fiber reinforced plastics (CFRP) is an effective technique to counteract both problems. Externally bonded reinforcement (EBR) leads to a higher load capacity which counteracts overloading. The second effect is that EBR causes a tension release from the interior reinforcement which can be affected by aging phenomena like fatigue or corrosion. In this case the application of EBR can extend the lifetime of a reinforced concrete bridge. For the prediction of the service life prolongation the distribution of forces between interior steel and EBR has to be exactly known. The change of the force distribution can be observed in combined double shear and pull-out tests under cyclic loading and described with a model taking the different bond stress slip behaviour of both reinforcement types into account.

EXPERIMENTAL TESTS

Test Setup

The cyclic tests are carried out in a hydraulic testing machine with pulsator for applying sinusoidal loads. The test setup is designed as a double shear test with near end support combined with a pull-out test at four steel bars under cyclic loading. The concrete test specimen have a length \(l = 1.35\) m and a square cross section with an edge length \(a_c = 25\) cm. The two externally bonded CFRP strips have a length \(l_L = 1.95\) m, a width \(b_L = 50\) mm and a thickness \(t_L = 1.4\) mm and the four steel bars have a diameter \(d_s = 16\) mm. The load is applied over a load distribution where the CFRP strips are applied over a clamping device and the steel bars are connected with a bolt nut over Belleville washers and spherical bearings. The concrete body is guided on roller bearings and compressed by a load plate connected with a threaded rod which runs bondless in a tube in the middle of the specimen. The CFRP strain is measured with a chain of strain gauges and the slip is measured with displacement transducers. The steel strains are measured with additional strain gauges on the steel bars. The strain gauges are mounted on the steel bars inside the concrete under the load plate in a bondless area. The strain gauges and the rebar are covered with a silicone layer and wrapped with a tape.
Materials

The experimental tests are carried out with CFRP strips with an average elastic modulus $E_L$ of 170,000 N/mm² and a cross section $A_L = 2 \times 1.4 \times 50$ mm². The adhesive is a two-component epoxy resin filled with rock flour. The thickness of the adhesive layer is set to 1 mm. The compressive strength of the epoxy after 7 days curing at 20 °C and 65 % humidity is 85.5 N/mm² the flexural strength is 46.1 N/mm² and the bulk density is 1.74 kg/dm³.

The specimen for the combined shear and pull-out test are made of concrete with a strength class C 20/25 according to Eurocode 2. The compressive strength $f_{cm}$ is determined in compressive tests on sample cubes with an edge length of 150 mm after 28 days to 39.0 N/mm². The tensile strength at the surface $f_{ctm, surf} = 1.4$ N/mm² is measured corresponding to EN 1542. The interior reinforcement steel is a B 500 B according to DIN 488 with a yield stress $f_{yd} = 500$ N/mm² and an elastic modulus $E_s = 210,000$ N/mm².

Test Procedure and Results

Four combined double shear pull-out tests are carried out under sinusoidal cyclic loading with a constant upper and lower load. The frequency was set to 4 Hz. The load level is adjusted to the static debonding load of the externally bonded reinforcement $F_{LRd}$, which has been determined in static double shear tests without interior reinforcement. In the first two tests of series A the lower load level of the externally bonded reinforcement is set to 15 % of the static debonding load. In Series B the lower load level is set to 34% and 33%. In the first tests of each series the upper load level is set to a value, where no damage occurs and two million load cycles were tested. In the second test the upper load was increased until decoupling occurs. Table 1 is showing the values of the lower and the upper load level $F_{UL}/F_{LRd}$ and $F_{OL}/F_{LRd}$ during the tests and the number of load cycles $N$ reached.

<table>
<thead>
<tr>
<th>Test</th>
<th>Lower Load $F_{UL}/F_{LRd}$</th>
<th>Upper Load $F_{OL}/F_{LRd}$</th>
<th>Load Cycles $N$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A 1</td>
<td>0.15</td>
<td>0.32</td>
<td>2,000,000</td>
</tr>
<tr>
<td>A 2</td>
<td>0.15</td>
<td>0.64</td>
<td>6,720</td>
</tr>
<tr>
<td>B 1</td>
<td>0.34</td>
<td>0.51</td>
<td>2,000,000</td>
</tr>
<tr>
<td>B 2</td>
<td>0.33</td>
<td>0.59</td>
<td>428,569</td>
</tr>
</tbody>
</table>

![Figure 1 Experimental setup for combined double shear pull-out test](image1)

![Figure 2 Strain at upper load level dependant on the number of load cycles during test A2 and B2](image2)
The redistribution of the forces during the tests A 2 and B 2 can be observed by the strain measurements, shown in Figure 2. The decoupling caused by the crack formation in the concrete substrate along the bonded length leads to a decrease of the CFRP strain $\varepsilon_L$ and an increase of the steel strain $\varepsilon_S$.

**MODEL AND CALCULATION**

The following model allows the determination of the force distribution between internal and external reinforcement under cyclic loading. With the crack propagation rate $da/dN$ the slip increase per load cycle $ds/N$ is calculated. The increasing slip of the externally bonded reinforcement leads to a redistribution of forces. The force in the EBR is decreasing while the force in the interior reinforcement is rising. The increase of the steel strain can be calculated using the formulation for the crack propagation rate according to Carloni (2015) and Leusmann (2015) together with the engineering approach for the force distribution after Zehetmaier (2006).

**Crack Propagation**

The bond behaviour of EBR under cyclic loading can be observed in cyclic shear tests. In a well-matched strengthening system, consisting of CFRP and adhesive, bond damage mostly occurs in form of cracking in the concrete substrate along the adhesive joint. In cyclic shear tests the crack formation can be observed with suitable measurement technology like optical measurement, see Carloni (2012) and Carloni (2013), or with a chain of strain gauge, see Leusmann (2015). The crack formation leads to an increase of the CFRP strain in the bonded area until a maximum value at the loaded end is reached. After that the stress transfer length remains constant and the S-formed strain distribution moves from the loaded end over the bonded length. Figure 3 shows schematically the experimental setup and the development of the CFRP strain $\varepsilon_L$.

![Figure 3 schematically experimental setup and development of CFRP strain $\varepsilon_L$.](image)

The crack propagation rate depends on the upper and lower load level and the static debonding load and can be determined with equation (1) after Carloni (2015).

$$\frac{da}{dN} = \frac{1}{N^*} \left( \frac{F_L^O - F_L^U}{\epsilon_1 \cdot (\Delta F_{L,Rd} - F_L^U)} \right)^k$$

The equation is applicable when the upper load $F_L^O$ exceeds the elastic limit $F_{L,Rd, fat_1}$, see German Committee for Structural Concrete (2012).

$$F_{L,Rd, fat_1} = 0.348 \cdot f_{ctm, surf}^{\frac{1}{4}} \cdot F_{L,Rd}$$

For smaller loads the CFRP to concrete bond responds perfectly elastic and there is no damage. Moreover, the upper load $F_L^O$ must not exceed the static debonding load $F_{L,Rd}$. In this case an immediate complete decoupling of the EBR would occur. Within these limits, the crack propagation can be calculated in mm for each load step $dN$ with equation (1).

**Force Distribution**

With the model from Zehetmaier(2006) the force distribution between steel rebar and EBR can be determined under static load. In agreement with force and moment balance and strain and slip distribution in a cracked cross...
section, the force contribution in the concrete, steel reinforcement bars and EBR are calculated with an iterative procedure. Initially a linear course of the strain distribution and the slip or the crack opening is assumed. The slip and strain states are coupled with coefficients \( k_{i} \) which are calibrated on measurement data depending on the reinforcement stiffness \( E_{lt} \), \( E_{st} \) and the concrete strength. Figure 4 shows the reinforced concrete cross section and the idealized distribution of strain, and displacement as well as the internal forces.

\[
\frac{\delta_{lk}}{\varepsilon_{s}} = \frac{d - x}{d_{L} - x} = \frac{2s_{L} - s_{L1} - \left( s_{L} - s_{L1} \right)^{2}}{s_{L0} - s_{L1}} \left( \frac{d_{L} - x}{d_{L} - x} \right) \left( \frac{\alpha_{s} + 1}{\alpha_{s} + 1} \right) \frac{s_{L}}{k_{si}} \frac{d_{L} - x}{d - x}
\]

(3)

For design applications Zehetmaier has developed a simplified model that allows the determination of the force distribution using the bond coefficient \( \delta_{lk} \) according to equation (3) which is the ratio of CFRP strain \( \varepsilon_{L} \) to steel strain \( \varepsilon_{s} \), adjusted for the distance of the reinforcement strands to the neutral axis, see Zehetmaier (2006). CFRP and steel strain \( \varepsilon_{L} \) and \( \varepsilon_{s} \) can also be determined with respect to the bond stress slip relationship of each reinforcement type using the CFRP slip \( s_{L} \) adjusted with the coupling coefficient \( k_{i} \). A method for the identification of \( s_{L} \) is shown in Zehetmaier (2006). The necessary parameters are listed in Table 2.

The bond coefficient \( \delta_{lk} \) is used for the calculation of the CFRP strain \( \varepsilon_{L} \) with equation (4) from the CFRP strain assuming a plane strain state \( \varepsilon_{L}^{II} \).

\[
\varepsilon_{L} = \left( 1 + \frac{E_{L} \cdot A_{L}}{E_{s} \cdot A_{s}} \frac{z_{L}}{z_{s}} \frac{d - x}{d_{L} - x} \right) \cdot \delta_{lk}
\]

(4)
Figure 5 shows the bond coefficient $\delta_{Lk}$ and the bond stress slip behaviour for the internal reinforcement according to Eligehausen (1983) and for the EBR corresponding to Holzenkämpfer (1994).

The development of $\delta_{Lk}$ can be described very well considering the bond stress slip relationships of steel and EB reinforcement. First the EBR to concrete bond reacts stiffer. This leads to a start value for $\delta_{Lk}$ smaller than 1. Subsequently the bond stress of EBR exceeds for a short section the bond stress of the cast-in reinforcement and $\delta_{Lk}$ reaches a value greater than 1. However, after reaching the maximum bond stress value $\tau_{L1}$ the EBR to concrete bond starts softening while the bond stress of the internal reinforcement is rising further. This is resulting in a decreasing bond coefficient $\delta_{Lk}$.

**Calculation Method**

The test results can be recalculated taking the crack propagation after equation (1) and the bond coefficient after equation (3) into account using the following procedure for every load step. In a first step the crack growth $da$ is calculated with equation (1). The resulting slip increase is determined with equation (5) and the actual slip $s_{L,i}$ with equation (6). After that the actual bond coefficient $\delta_{Lk,i}$ is calculated with equation (7) and the actual CFRP strain $\varepsilon_{L}$ can be calculated from $\varepsilon_{L,II}$ with equation (4). The results of the recalculation are shown in Figure 6.

\[
\begin{align*}
    ds_L &= da \cdot \frac{F_{L}}{E_L \cdot A_L} \\
    s_{L,i} &= s_{L,i-1} + ds_L \\
    \delta_{Lk,i} &= \frac{e_{L,i-1} \cdot d - x}{d_L - x} \\
    \sqrt{8k_s \sqrt{f_{cm}}} &\left( \frac{s_{L,i}}{E_s d_s} \right) \left( \frac{d_L - x}{k_{SL} d_L - x} \right)
\end{align*}
\]

![Figure 6](image)

Figure 6 Measured and calculated strain at upper load level vs. number of load cycles during test A2 and B2

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Abbr.</th>
<th>unit</th>
<th>value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Youngs modulus of CFRP</td>
<td>$E_L$</td>
<td>N/mm²</td>
<td>170,000</td>
</tr>
<tr>
<td>CFRP cross section</td>
<td>$A_L$</td>
<td>mm²</td>
<td>140</td>
</tr>
<tr>
<td>Youngs modulus of Steel</td>
<td>$E_s$</td>
<td>N/mm²</td>
<td>200,000</td>
</tr>
<tr>
<td>Steel cross section</td>
<td>$A_s$</td>
<td>mm²</td>
<td>804</td>
</tr>
<tr>
<td>Steel rebar diameter</td>
<td>$d_s$</td>
<td>mm</td>
<td>16</td>
</tr>
<tr>
<td>Maximum CFRP bond stress at $\tau_{L1}$</td>
<td>$s_{L1}$</td>
<td>mm</td>
<td>0.039</td>
</tr>
<tr>
<td>Maximum CFRP Slip</td>
<td>$s_{L0}$</td>
<td>mm</td>
<td>0.201</td>
</tr>
<tr>
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<td>2</td>
</tr>
<tr>
<td>Steel bond parameter</td>
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<td>-</td>
<td>0.25</td>
</tr>
<tr>
<td>Coupling coefficient</td>
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<td>-</td>
<td>0.844</td>
</tr>
<tr>
<td>Crack propagation factor</td>
<td>$k$</td>
<td>-</td>
<td>23.4</td>
</tr>
</tbody>
</table>
The slip increase of the interior steel caused by fatigue damage is very low at the observed level and therefore it is neglected in the recalculation procedure. The factor for the distance of the reinforcement strands to the neutral axis \( d - x \) and the relation of the lever arms \( \frac{z_h}{z_c} \) is set to 1. The parameter \( c_1 \) for the determination of the crack propagation rate is fitted to a value of 0.35 for test A2 and to 0.38 for test B2. Other necessary parameters are listed in Table 2.

CONCLUSIONS AND PERSPECTIVE

The evolution of the force distribution between internal and external reinforcement under fatigue loading can be observed in combined shear and pull out tests under cyclic loading. The test results can be recalculated with the method presented in this paper taking into account the crack propagation and the bond behavior for both reinforcement types. Calculated and experimental results show a good correlation. The above given equations can also be used for the calculation of the force distribution at the end anchorage of CFRPs externally bonded to RC beams. The presented method above is a first step towards a model for the prediction of the service life prolongation of RC beams subjects to cyclic loading. For a precise analyse of a full strengthened rc beam a method for the consideration of debonding between cracks has to be developed. Furthermore creeping and fatigue in the concrete compression zone, bond creeping and the influence of cyclic loading on tension stiffening has to be considered.

REFERENCES


SHORT AND LONG TERM PERFORMANCE OF CFRP LAMINATE PRODUCED USING FURFURYL ALCOHOL BIO RESIN FOR CONCRETE RETROFITTING

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ABSTRACT

Externally-bonded fibre-reinforced polymer (FRP) sheets and plates have grown in popularity for structural retrofitting due to their high-strength-to-weight ratio, corrosion resistance and flexibility; however, they remain an unsustainable solution. Furfuryl alcohol resins, which are fully-derived from corn cob and sugarcane, represent a potential alternative to conventional petroleum-derived epoxy resins, although insufficient research has been done to verify their long-term durability – one of the most critical aspects due to the harsh environmental conditions experienced by infrastructure. In this article, the short- and long-term performance of the carbon fibre and furfuryl alcohol resin composites were examined for externally-bonded retrofitting applications. Long-term conditioning consisted of submersion in a 3.5%-by-weight saline solution at 23°C, 40°C and 50°C for up to 240 days. Sixty-five tension FRP coupons and thirty-nine FRP-bonded notched concrete prisms and cylinders were tested. The FRP tensile strength and modulus were 860MPa and 92.8GPa, respectively. After conditioning, the lowest strength and modulus retention were 81% and 87%, respectively. The FRP-bonded notched-concrete prisms demonstrated a normalized shear stress retention ranging from 88% to 104%. Fluctuations were observed throughout the conditioning periods due to the combined effects of the FRP, concrete and bond. After conditioning, the maximum stress experienced by the externally-bonded FRP was 54% of its conditioned strength.

KEYWORDS

Furfuryl alcohol, bio-resin, bond, FRP, durability, concrete, temperature, carbon fibre.

INTRODUCTION

Over the past decade, the application of externally-bonded fibre-reinforced polymer (FRP) sheets and plates for structural retrofitting has grown into a well-established technology due to its high-strength-to-weight ratio, corrosion resistance and flexibility; however, the main challenge in terms of widespread application is durability. Infrastructure is exposed to harsh environmental conditions, such as de-icing salts, freeze-thaw cycles and extreme heat and humidity. These conditions cause degradation of the FRP as well as the interface.

Research and field application of externally-bonded FRP to concrete beams typically involve the use of conventional FRP systems, such as those manufactured with carbon fibre and epoxy. Although the carbon fibers are inert, the resin system and the bond can be greatly affected by humidity as well as thermal and chemical conditions. These conditions have been simulated in laboratory environments (Rivera and Karbhari 2002; Cromwell et al. 2010; Robert and Fam 2012), including a proposed accelerated aging protocols for concrete-FRP bond (ACI 440.9R 2015) based on bond test recommendations and research by Gartner et al. (2011).

Although there has been significant progress in terms of durability, conventional FRP systems do not present a long-term solution. They are fundamentally unsustainable due to their reliance on petroleum. To address this challenge, several studies have assessed the use of natural thermosetting polymers, which can be derived from corn, soybean and tung oil (Lu et al., 2005; Li et al. 2003; Casado et al. 2009) as well as their use in combination with natural fibres (O’donnell et al. 2004; Mak et al. 2015); however, natural fibres systems demonstrated insufficient strength and stiffness for rehabilitation applications.
Furthermore, most commercially-available natural resins are resin blends, where the majority of the blend consists of conventional petroleum-derived resin. One promising alternative is furfuryl alcohol resin, which is fully derived from corncobs and sugarcane. Its derivative is furfural, which has one of the highest yields per ton of dry biomass at 220kg for corncob. It is also an agricultural by-product with Canada producing over 1.4 million tons of corn cob waste per year at approximately 10 cents/kg.

Structurally, furfuryl alcohol resin has been reported to have a similar response as epoxy when used in conjunction with fibreglass as a composite and for concrete cylinder confinement (Fam et al. 2013; Eldridge and Fam 2014a,b). Due to the required high stiffness of concrete retrofits, carbon fibre is favoured over fibreglass; however, limited research has been done on the application of furfuryl alcohol resin-impregnated carbon FRP.

This article presents an experimental program on the durability of furfuryl alcohol-based resin used in combination with carbon fibre with the goal of flexural concrete retrofitting. The study addresses short- and long-term performance of the novel FRP system as a pure composite and when externally-bonded to concrete. The long-term exposure was conducted in a 3.5%-by-weight saline solution at 23°C, 40°C and 50°C for up to 240 days.

MATERIAL SPECIFICATIONS AND TEST REGIME

Materials

**Furfuryl alcohol resin** \((C_5H_6O_2)\): is a low-viscosity resin, commercially available as QuaCorr1001. It is derived from renewable resources such as sugar cane and corncobs. It has a dark-reddish to brown color, and has the following physical characteristics: a specific gravity of 1.22, viscosity of 300-600 cps and a flash point of 75.6°C. p-Toluenesulfonic acid monohydrate 97.5% was used at 3%-by-weight to cure the resin.

**High viscosity epoxy resin**: is an epoxy paste specifically designed for rehabilitation, commercially available as Sikadur® 30. The reported tensile strength and elastic modulus after 7 days of curing at room temperature are 24.8 MPa and 4.5 GPa, respectively. The reported \(T_g\) after 7 days curing at 45°C is 62°C.

**Concrete**: had a 28-day design compressive strength of 35 MPa with a maximum aggregate size of 19 mm.

**Specimen Fabrication**

**FRP Tensile Coupons**

The FRP laminates were manufactured using the wet lay-up process. A single layer of carbon fibre fabric was laid out on a flat surface and covered with furfuryl alcohol resin. A flexible spreader was used to disperse the resin until the fabric was fully saturated. To ensure a consistent thickness, the saturated layer of fabric was sandwiched between two high density polyethylene plates and weights were placed on top of the plates. After two weeks of room temperature curing, the sheets were cut to nominal dimensions of 254x25 mm using a wet tile saw. They were then post-cured at 200°C for three hours, as recommended by Ma et al. (1995).

Carbon-epoxy tabs, manufactured similar to the above-described FRP laminates, were adhered to the gripping region using high viscosity epoxy resin to reduce the potential of grip failure, as suggested by ASTM D3039 (2007). For conditioned specimens, tab adhesion was done at least seven days post-removal from the tank.

**Notched-concrete Prism FRP-Bond**

Concrete prisms of 102x102x356 mm\(^3\) dimension were cast. After 28 days, the prisms were cut at mid-span to mid-depth using a sliding stone saw of 3 mm width. The notched half of the prism represents the tension side of the block, which was sandblasted prior to adhering the FRP sheet.

FRP sheets were fabricated in the same manner as the FRP tension coupons; however, the FRP sheets were cut to a nominal dimension of 25x204 mm\(^2\) after two days. Upon full cure, high viscosity epoxy resin was applied to the concrete surface and the FRP sheet. An aluminum roller was used to apply the sheet onto the concrete. All prisms were cured at room temperature for 14 days prior to testing or conditioning.

**Conditioning**

The conditioning procedure consisted three salt water tanks with an initial salt concentration of 3.5%-by-weight at 23°C, 40°C and 50°C as per Bank et al. (2003). Screw-plug heaters were used in conjunction with an electric
temperature controller to maintain the temperature at 40°C and 50°C to a 1°C variation, whereas the 23°C tank was maintained at room temperature. Pumps with a 0.12 L/s flow rate were installed in the tanks to eliminate temperature differentials. A handheld thermistor was used to verify the temperature. An insulated sheet sealed the top of the tank to reduce evaporation and heat loss. Additional insulation surrounded the walls of the heated tanks.

After five months of conditioning, the 40°C tank heating element failed. Specimens in the tank were removed and stored temporarily in standard lab conditions. Due to the backorder of the failed mechanism, the specimens were placed back in the conditioning tank after 32 days and the remaining period of conditioning was completed.

**Test Setup**

**FRP Tensile Coupons**

FRP specimens were tested in tension according to ASTM D3039 (2007). An extensometer with a 25 mm gage length was used to measure the FRP strain. It was removed prior to specimen failure to ensure that it would not be damaged. The stress-strain response was extrapolated until failure. The linear stress-strain trend was verified with a strain gauge. Load was measured directly from the testing frame.

**Notched-concrete Prism FRP-Bond**

Prisms were tested in three-point bending with a span of 305 mm (Gartner et al. 2011; ACI 440.9R (2015)) at a loading rate of 0.25 mm/min (Figure 1(a),(b)). The load was applied using a 25 mm diameter steel cylinder. Load was measured from the testing frame, and the mid-span deflection was measured using a linear potentiometer.

The measured maximum load at failure was used to calculate the average interfacial shear strength (Gartner et al. 2011). For this calculation, the concrete compressive stress level at bond failure is assumed to be small and as such, stress is linear and the neutral axis is stable at mid-depth (Figure 1(c)). Based on the simple geometry, interfacial shear strength (τ) at bond failure, is calculated as a function of the measured ultimate load (P), where \( L \) is the span, \( h \) is the prism height, \( w \) is the CFRP laminate width and \( S \) is the CFRP laminate length as follows:

\[
\tau = \frac{3PL}{5hwS}
\]

ASTM C39 (2015) was used to determine the concrete compressive strength at each conditioning stage.

![Figure 1 Notched-concrete prism: (a) test set-up, where AE is aluminium extender and LP is linear potentiometer, (b) schematic of specimen and (c) idealized internal forces and shear](image)

**EXPERIMENTAL RESULTS AND DISCUSSION**

**Failure Behaviour**

**FRP Tensile Coupons**

A summary of the tensile test results is shown in Table 1. To determine the tensile properties, the average thickness of 1.78±0.13 mm and average width of 25.34±0.12 mm were used. All failure modes were acceptable, either as explosive or delamination within the middle of the gauge length. No trends were observed between failure mode, conditioning temperature and conditioning period. All coupons showed a linear stress-strain response until failure.
Specimen ID | Environmental Condition | Temp. (°C) | Period (days) | Ultimate tensile strength (MPa) | Tensile strength retention (%) | Tensile modulus (GPa) | Tensile modulus retention (%) |
<table>
<thead>
<tr>
<th></th>
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<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
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<tbody>
<tr>
<td>FA-Control</td>
<td>Control, dry</td>
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<td>-</td>
<td>860</td>
<td>92.8</td>
<td>5.84</td>
<td>-</td>
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<tr>
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<td></td>
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<td>722</td>
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<td>83.9</td>
<td>70.3</td>
<td>105</td>
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<td>84.8</td>
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<td>712</td>
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<td>82.8</td>
<td>85.7</td>
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<td>86.2</td>
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<td>740</td>
<td>49.7</td>
<td>86.1</td>
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<td>92</td>
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<td>699</td>
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<td>81.2</td>
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<td></td>
<td>30</td>
<td>800</td>
<td>81.3</td>
<td>93</td>
<td>85.9</td>
<td>92.6</td>
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<td>98.6</td>
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<td>92.1</td>
<td>94</td>
<td>101.4</td>
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<tr>
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<td>240</td>
<td>710</td>
<td>57.5</td>
<td>82.5</td>
<td>81.1</td>
<td>103.8</td>
</tr>
</tbody>
</table>

**Notched-concrete Prism FRP-Bond**

A summary of the notched-concrete prism test results is shown in Table 2. The interfacial shear strength was calculated based on the ultimate load using Eq. (1). The normalized shear strength was done so by dividing by their respective square roots of \( f' \). Concrete cylinders were conditioning to determine the compressive strength.

Table 2 Bond test matrix and results, where all values are based on an average of three repetitions

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Environmental Condition</th>
<th>Temp. (°C)</th>
<th>Period (days)</th>
<th>( f' ) (MPa)</th>
<th>Ultimate Shear Strength (MPa)</th>
<th>Normalized Shear Strength Retention (%)</th>
</tr>
</thead>
<tbody>
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<td>Notch-Control</td>
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</tr>
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<td>-</td>
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<td></td>
<td>30</td>
<td>34.1</td>
<td>5.36</td>
<td>0.41</td>
<td>0.85</td>
</tr>
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<td>39.9</td>
<td>5.79</td>
<td>0.21</td>
<td>1.00</td>
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<td></td>
<td>120</td>
<td>33.9</td>
<td>5.79</td>
<td>0.21</td>
<td>1.00</td>
</tr>
<tr>
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<td></td>
<td>240</td>
<td>33.8</td>
<td>5.57</td>
<td>0.62</td>
<td>0.96</td>
</tr>
<tr>
<td>FAC-30-40</td>
<td></td>
<td>30</td>
<td>38</td>
<td>5.92</td>
<td>0.41</td>
<td>0.96</td>
</tr>
<tr>
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<td>60</td>
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<td>5.40</td>
<td>0.44</td>
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<td>6.66</td>
<td>0.33</td>
<td>1.00</td>
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<td>FAC-30-50</td>
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<td>30</td>
<td>32</td>
<td>5.08</td>
<td>0.41</td>
<td>0.90</td>
</tr>
<tr>
<td>FAC-60-50</td>
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<td>60</td>
<td>32.8</td>
<td>5.37</td>
<td>0.17</td>
<td>0.94</td>
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<tr>
<td>FAC-120-50</td>
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<td>40.7</td>
<td>5.89</td>
<td>0.65</td>
<td>0.92</td>
</tr>
<tr>
<td>FAC-240-50</td>
<td></td>
<td>240</td>
<td>35.5</td>
<td>5.94</td>
<td>0.61</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Two failure modes are acceptable: adhesive and cohesive bond failure. Adhesive failure is defined by traces of cement paste and fine aggregates on the laminate, whereas cohesive failure is defined by a thick layer of concrete and coarse aggregates on the laminate. Concrete shear failure is not acceptable, as it does not provide a gauge of bond performance of the system. Cohesive bond failure was the primary failure mode specimens and was consistent regardless of the conditioning regime. Due to the constancy of cohesive failure, the bond between the furfuryl alcohol resin and the high viscosity epoxy resin was excellent as interface never governed failure.

**Effect of Conditioning**

**FRP Tensile Coupons**

The average tensile strength and modulus of unconditioned specimens were 860±110 MPa and 93±6 GPa, respectively. After 240 days, the strength retention at 23°C, 40°C and 50°C was 83%, 81% and 82%, respectively,
whereas the tensile modulus retention at 23°C, 40°C and 50°C was 92%, 94% and 87%, respectively. The variations in strength over time can be observed in Figure 2(a).

![Figure 2](image_url)

**Concrete**

The unconditioned concrete compressive strength was 33MPa. Throughout the conditioning phases, the concrete strength varied from -3% to +34%. This can be attributed to two opposite effects: moisture allowed for further curing and the continuation of the hydration process, which results in strength gain; conversely, calcium leaching resulted in an pH imbalance between the cylinders and the water, which resulted in the disintegration of Calcium-Silicate-Hydrate (CSH) and a subsequent strength loss. The combined opposing effects can be seen in Figure 2(b).

**Notched-concrete Prism FRP-Bond**

The unconditioned notched-concrete prisms showed a normalized shear stress of 0.97 MPa. After 240 days, the normalized shear stress retention at 23°C, 40°C and 50°C was 100%, 104% and 103%, respectively. However, this does not always represent the highest strength loss. For example, the lowest strength retention at 23°C and 40°C were 88% and 97%, respectively, at 60 days of conditioning, whereas the lowest strength retention at 50°C was 93% at 30 days of conditioning. The fluctuations observed in strength are due to the combined effects of the changing FRP, concrete and bond strength. Further research is necessary to determine the exact implications. The variation in normalized shear stress is presented in Figure 2(c).

To determine the efficiency of the FRP system, the stress in the FRP at bond failure for the notched-concrete prisms was compared to the strength of the tensile FRP coupons with the same conditioning regime. In an unconditioned state, bond failure occurred when the FRP reached 36% of its strength. After 240 days, the percentage of stress in the FRP compared to the strength at 23°C, 40°C and 50°C was 44%, 54% and 47%, respectively. Fluctuations similar to that of the normalized shear stress were observed throughout the conditioning period; however, the maximum observed value was 54% of the strength after 240 days of conditioning at 40°C. The variation in percentage strength is presented in Figure 2(d).

**CONCLUSION**

This study examined the short- and long-term performance of carbon fibre and furfuryl alcohol resin composites aimed at concrete retrofitting. Long-term conditioning occurred throughout 240 days at 23°C, 40°C and 50°C in a 3.5%-by-weight saline solution. The following conclusions were drawn:
1. The FRP tensile strength and modulus are 860 MPa and 92.8 GPa, respectively. After 240 days at 23°C, 40°C, and 50°C, strength retention was 83%, 81% and 82%, respectively, and modulus retention was 92%, 94% and 87%, respectively.

2. Throughout the conditioning period, concrete strength fluctuated by -3% to +34% due to moisture, which allowed for additional curing, and calcium leaching, which caused disintegration of the cement paste.

3. Throughout the conditioning period, the normalized shear/bond stress at failure ranged from 88% to 104%. This value fluctuated due to the combined effects of the FRP, concrete and bond.

4. Taking FRP deterioration into account, the maximum stress experienced by the externally-bonded FRP was 54% of its strength when bond failure occurred after 240 days of conditioning at 40°C. This increased from 35% when tested in an unconditioned state.

ACKNOWLEDGEMENTS

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ABSTRACT

Increase in traffic volume and load on highway bridges requires replacement of the bridge superstructure or adoption of a structural strengthening system. Strengthening of existing prestressed concrete bridges with conventional construction materials results in an increase in dead load. In addition, it may also result in decrease in bridge clearance. The use of Carbon fiber reinforced polymer (CFRP) composites is considered a practical alternative for flexural strengthening of prestressed concrete bridge girders. However, the effectiveness of the CFRP depends on several factors, which have not been quantified well. AASHTO type II prestressed girder is one of the most popular girders carrying highway bridge decks, in the U.S.A. This paper presents a finite element analysis of CFRP strengthened AASHTO type II prestressed girder. Several variables were investigated: concrete strength, ratio of prestressing strands, and ratio of CFRP composites. For CFRP strengthened PC girders, the results of this investigation suggest that for the same amount of CFRP, the lower the ratio of prestressing strands, the higher the percentage increase in flexural strength. In addition, strengthened girders exhibit comparable ductility as un-strengthened PC girders.

KEYWORDS

Finite element analysis, CFRP composites, flexural strength, prestressed girders, highway bridges.

INTRODUCTION

According to the Transportation Research Board (TRB) reports in United States, over 40% of the nation’s bridge are structurally deficient and functionally obsolete which are in need of repair, strengthening or replacement. Compared with replacement of the outdated and deficient bridge components which could be economically unfeasible, using FRP systems in the form of “external” reinforcement to strengthen the deficient prestressed structures may lower the costs and be more efficient for prestressed concrete bridges (Mayo et al. 1999).

During the past two decades, several experimental researches have been conducted to investigate the compact of CFRP on the behaviour of concrete members. Aboutaha (1999) did an experimental investigation on the application of CFRP for strengthening a damaged AASHTO type II prestressed concrete girder. The experimental results showed that CFRP could restore the stiffness and increase the strength of the damaged prestressed concrete girders. Mayo et al. (1999) performed several in-situ field tests using CFRP to strengthen a simple span reinforced concrete beam. Based on the test results, they concluded that the CFRP could increase the flexural strength of reinforced concrete beam. Ross et al. (1999) investigated the effect of the reinforcement ratio on the impact of the externally strengthened beams with CFRP. They found that for the light reinforcement ratio, the tensile strength was controlled by the concrete in the tension zone and the failure of the tested beam was the debonding of the CFRP. The CFRP could increase the load capacity of the beam effectively. For the heavy reinforcement ratio, the failure mode was the crushing of the concrete on the compression zone. CFRP had less impact on the load capacity of the beam.

Moreover, finite element models have been increasingly used as a numerical tool to study the behavior of concrete structures with CFRP in the past decade. Hu et al. (2004) did a nonlinear finite element analysis of reinforcement concrete beam strengthened with FRP. Nonlinear constitutive equations for the FRP are coded in FORTRAN language as a subroutine and linked to the ABAQUS program. Perfect bond was assumed during the modeling, 4-node shell element was used to model the FRP strips (Hu et al. 2004). Jose (2005) modeled a FRP strengthening
system on the bridge with ABAQUS. The FRP material was assumed to be homogeneous, linearly elastic and orthotropic. The 2-D analysis was considered because the FRP strips have two dimensions much larger than the third one. The bending stiffness and the stresses in the out of plane direction were neglected. The ABAQUS tool called “Skin Reinforcement” was used to model the FRP strips. For “Skin Reinforcement”, only one skin can be placed on a surface of an element, which means that the skins cannot overlap. To model the cases for 2 layers of strips, the same section used for 1 layer cases was applied, but doubling the value for the thickness of the section. Perfectly bond between the concrete surface and FRP was assumed. Membrane elements were used to model the FRP (Jose, 2005). Charalambidi et al. (2012) modeled a RC column with FRP jacket with ABAQUS. The material properties of FRP was assumed to be homogeneous, linearly elastic and orthotropic. Also, perfectly bond between the concrete surface and FRP was assumed and membrane elements were used to model the FRP jacket (Charalambidi et al., 2012).

The aim of this study is to pursue a finite element analysis of CFRP strengthened AASHTO type II prestressed girder using ABAQUS/CAE. The verification of the FEA model was a prestressed AASHTO Type II girder tested by Aboutaha (1999). In the finite element analysis, several variables were investigated: concrete strength, ratio of prestressing strands, and ratio of CFRP composites.

FINITE ELEMENT MODEL

Element Types

In order to model the behavior of concrete, an 8-node linear brick, reduced integration with hourglass control element (C3D8R) was used. The brick elements in ABAQUS are used for linear analysis and nonlinear analysis involving contact and plasticity, and they are available for stress analyses. C3D8R element has 8 nodes and each node has 6 degrees of freedom (displacement and rotation in x, y, z-directions). The C3D8R elements are first-order elements, so the strain operators provides constant volumetric strain throughout the element. The reduced integration for this element uses a lower-order integration to form the element stiffness, which reduce running time in 3-D modeling. The hourglass control prevent the uncontrolled distortion of the mesh. Prestressing strands were modeled using a 6-node linear triangular prism (C3D6) element with an equivalent round section. The C3D6 element has 6 nodes and each node has 6 degrees of freedom (displacement and rotation in x, y, z-directions). The stirrups in the prestressed girder were modeled with 2-node truss elements (T3D2). The truss elements support loading along the axis of the element without moments or forces perpendicular to the centerline. The CFRP strips are modeled with shell elements (S4R). Shell elements are usually used to model structures which can be analyzed with plane stress theory. S4R is a 4-node with reduced integration, general purpose, and finite-membrane-strain shell element. The uniformly reduced integration avoid shear and membrane locking.

Material Model

The accuracy of the FEA results depend heavily on the material properties used in the model. The constitutive relations of the concrete, strand, CFRP are discussed in this section.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Dilation Angle</th>
<th>Eccentricity</th>
<th>fb0/fc0</th>
<th>K</th>
<th>Viscosity Parameter</th>
<th>Poisson’s Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value</td>
<td>31</td>
<td>0.1</td>
<td>1.16</td>
<td>0.667</td>
<td>0.001</td>
<td>0.2</td>
</tr>
</tbody>
</table>

Concrete material properties

The concrete material model used in this FEA analysis was the “Concrete Damaged Plasticity (CDP)” model in ABAQUS/CAE. The constitutive relation of the concrete was based on AASHTO LRFD Bridge Design
Specifications and previous research studies as shown in Figure 1. The elastic modulus is assumed to be the same for both compression and tension behaviour (Figures 1 and 2) which was calculated based on the AASHTO LRFD Bridge Design Specifications. The elastic range for the compressive behaviour was assumed to be 0.3fc’. The Thorenfeldt et al. (1987) constitutive model for concrete in compression was used which was applicable for concrete strength from 2000 psi to 18000psi. The parameters used in the CDP model are summarized in Table 1. The concrete tension stiffening was considered and a simplified curve was used in this model as Figure 3.

Strand material properties

The prestressing strand are 270ksi, low-relaxation strand. The material properties of the strand are based on the model provided by PCI manual (6th Edition) as Figure 4.

![Figure 4 PCI Standard 270 ksi. Low-relaxation strand constitutive model (PCI Manual)](image)

CFRP material properties

The prestressed concrete girder was strengthened using Mitsubishi’ Replark system (Replark™ Type 30 (MRK-M2-30)). The material properties provided by the manufactures are as following:

<table>
<thead>
<tr>
<th>Properties</th>
<th>Width (in)</th>
<th>Thickness (in)</th>
<th>Tensile Modulus (ksi)</th>
<th>Ultimate Strain</th>
<th>Tensile Strength (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value</td>
<td>12</td>
<td>0.0066</td>
<td>33360</td>
<td>0.015</td>
<td>493.128</td>
</tr>
</tbody>
</table>

FRP material has been assumed to behave as a linearly elastic and orthotropic material. The “lamina” option for the elastic material behavior was chosen, which describe the orthotropic elasticity in plane stress. For this type of material, ABAQUS requires the longitudinal, transverse and shear modulus of elasticity. The manufactory of Replark™ Type 30 (MRK-M2-30) only provided the longitudinal modulus of elasticity, the transverse and shear modulus were assumed based on the average properties of carbon fibers (Jose, 2005).

Contact Model

For the contact behavior between prestressing strand and concrete, several methods are available. The most common ways are perfect bond which can be achieved by using embedded or tied constraints, non-linear spring elements, and surface friction interaction model. The embedded or tied constraints are easy to model, however, they cannot model the bond-slip phenomena at the strand end region which exists in the prestressing girder after the strand release. The non-linear spring element can model the bond-slip behavior, however, this method is time consuming for 3-D modeling. So, the bond between prestressing strand and concrete was modeled with a surface-to-surface contact element with tangential and normal behavior. The tangential behavior was modeled with penalty friction formulation with. The friction coefficient was assumed to be 1.4 after calibration. The default value of the maximum elastic slip in ABAQUS/CAE was used, which was 0.005. The normal behavior was modeled with “Hard” Contact which minimize the penetration of the slave nodes into the master surface at the onset of the contact and prevent the transfer of tensile stresses with the contact surface. The contact between CFRP strips and concrete surface are assumed fully-bonded without bond failure. The different layers of CFRP were modeled with different thickness of the CFRP, such as two layers of CFRP were modeled with a CFRP strip with a thickness of 0.0132 in.
Boundary Conditions and Loading

Due to symmetry in both x-direction and z-direction, only quarter of each girder was modeled. To model the x-direction symmetry, the girder was cut with a vertical plane in y-z plane. The displacement in x-direction and rotation around y, z-direction were constrained with a roller supports (U1=UR2=UR3=0). To model the z-direction symmetry, the girder was cut with a vertical plane in x-y plane. The displacement in z-direction and rotation around x, y-direction were constrained with a roller supports (U3=UR1=UR2=0). The girder was supported with roller supports. The displacement in y-direction at the support plate was restricted (U2=0). To avoid the load concentration, the external load was applied in y-direction as a pressure on a rigid load plate which was fully bonded with the concrete surface. The prestressing force was applied in the initial step as an initial condition using the PREDEFINED FIELD of ABAQUS/CAE.

VERIFICATION OF THE FE MODEL

The FEA model was verified against 2 different experimental results from Aboutaha (1999). Aboutaha tested one prestressed AASHTO Type II girder under two different tests as Figure 5. A comparison between the load-deflection curves obtained by Aboutaha (1999) and the load-deflection curves obtained by the FEA model with ABAQUS/CAE are shown in Figure 6 and Figure 7. It can be seen that the FEA model can reasonably estimate the ultimate capacity of the prestressed concrete girder.

RESULTS AND DISCUSSIONS

In this section, the effects of reinforcement ratio (strand ratio, SR), concrete compressive strength, and layers of CFRP strips on the ultimate flexural strength of AASHTO Type II prestressed concrete girders are investigated. The load-deflection results for different layers of CFRP (0, 1, 2, and 3) with different concrete compressive strength and reinforcement ratio are shown in Figure 8.
From the load-deflection and ultimate capacity enhancement ratio above, we can find that, CFRP has almost no impact on the initial flexural stiffness of the prestressed concrete beam. As the strand ratio equals to 0.3%, the ultimate flexural capacity of the prestressed girders increased with the increase of the amount of CFRP. As the strand ratio equals to 0.6% and 0.9%, the amount of CFRP has little impact on the ultimate flexural capacity of the prestressed girders. This might be caused by the following two reasons:

1. As the amount of CFRP increases, the depth of the neutral axis increases, which leads to a smaller moment arm between tension and compression force. Consequently, the flexural capacity will not increase much.
2. Compared to the force in the prestressing strand, the internal forces in the CFRP are relatively small, therefore, the impact of the CFRP on the flexural strength is very limited.

Based on these two reasons, the neutral axis and the tensile force provided by the CFRP compared with the total tensile force at ultimate state for each girder was tested. It was observed that the neutral axis decrease largely for several girders strengthened with CFRP (Girder case: fc'=5ksi, SR=0.3%, CFRP=0,1,2,3 layers). However, for most girders, the neutral axis didn’t change too much. The tensile force provided by different layers of CFRP for the girders (fc'=5ksi, SR=0.3%) are 5.671%, 9.14%, and 11.181%, respectively. The tensile force provided by different layers CFRP for the girders (fc'=10ksi, SR=0.3%) are 9.544%, 15.583%, and 19.746%, respectively. The tensile force provided by the CFRP for the girders (fc'=5ksi, SR=0.6% and 0.9%) are from 0.626% to 3.027%.

Figure 8 FEA load-deflection results for different layers of CFRP (0, 1, 2, and 3) with different concrete compressive strength and reinforcement ratios.
The tensile force provided by the CFRP for the girders (fc' = 10 ksi, SR = 0.6% and 0.9%) are from 0.988 % to 5.374%. Based on these FEA results, we can find that the force provided by the CFRP are too small compared with the force provided by the prestressing strand. So, the impact of the CFRP on the ultimate flexural capacity of the prestressed concrete girder are small. In addition, for the same amount of CFRP, the higher the amount of prestressing strands, the lower the increase in the flexural strength.

**CONCLUSIONS**

- For the AASHTO Type II girders, the CFRP has very limited impact on the initial flexural stiffness of the girders.
- For the same amount of CFRP, the higher the amount of prestressing strands, the lower the percentage increase in the flexural strength.
- The higher the ratio of CFRP to steel strand ratio the higher the increase in the flexural strength.
- The CFRP strengthened prestressed concrete girders exhibited good ductility, which were comparable to those of ordinary prestressed concrete girders designed according to the current AASHTO specifications.

**REFERENCE**


Aboutaha, R. S. (1999). "Structural Rehabilitation of Prestressed Concrete Bridge Girders Using CFRP Composites". *8th International Conference and Exhibition, Structural Faults and Repair*.


EXPERIMENTAL RESEARCH ON MASONRY WALL STRENGTHENED WITH LARGE RUPTURE STRAIN FRP

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ABSTRACT

In this study a type of FRP with large rupture strain known as PET (Polyethylene Terephthalate) FRP was employed to strengthen unconfined and confined masonry walls. Four full-scale masonry walls were designed and prepared, one was unconfined control specimen, one was confined control specimen, one was unconfined masonry wall strengthened with PET placed along two diagonals and vertical edges, and one was confined masonry wall strengthened with PET only placed along two diagonals with end anchorage. The PET was applied symmetrically on both sides of masonry walls. The failure of all specimens was triggered by the opening of diagonal cracks. The test results showed that the peak strength, the displacement at failure, the energy dissipation capacity and the ductility factor of strengthened unconfined masonry wall had 90%, 87%, 406% and 74% increase respectively compared with that of the unconfined control specimen, while for the strengthened confined masonry wall, the peak strength, the displacement at failure, the energy dissipation capacity and the ductility factor were 52%, 40%, 145% and 23% higher than that of the confined control specimen, respectively.

KEYWORDS

Masonry wall, large rupture strain (LRS) FRP, polyethylene terephthalate (PET), strengthen, cyclic loading.

INTRODUCTION

In the past two decades, a number of experimental research works have been carried out to investigate the in-plane behaviour of masonry walls strengthened with typical FRP materials. Different layouts of FRPs were applied either on double sides or single side of precracked or uncracked specimens. Results showed that the shear capacity and deformation capacity of uncracked walls were significantly increased (Schwegler 1994; Valluzzi et al. 2002; Holberg and Hamilton 2002; Chuang et al. 2003; Stratford et al. 2004; Santa-Maria et al. 2006; Alcaino and Santa-Maria 2008; Rahman and Ueda 2015, 2016; Lei et al. 2016), and the bearing capacity and ductility of damaged walls were restored or even exceeded the original capacity (Vandergrift et al. 2002; ElGawady et al. 2007; Santa-Maria and Alcaino 2011; Jing and Raongjant 2015). A little or none uneven response due to strengthening on single side was observed (Valluzzi et al. 2002; Holberg and Hamilton 2002; Chuang et al. 2003; Stratford et al. 2004; ElGawady et al. 2007; Jing and Raongjant 2015).

One of the important factors that influence the behaviour of strengthened walls is the reinforcement layout. It was found that the full surface coverage resulted in the highest bearing capacity but the least amount of deformability. The diagonal configuration was found more efficient in terms of shear capacity than the grid layout, but the latter offered a better stress redistribution and caused a less brittle failure (Schwegler 1994; Valluzzi et al. 2002; Santa-Maria et al. 2006; Alcaino and Santa-Maria 2008; Rahman and Ueda 2015, 2016; Jing and Raongjant 2015).

Since typical FRP materials such as CFRP, BFRP, and GFRP are stiff materials and vulnerable to debonding, failure of most test specimens were characterized by brittle failure across the masonry-FRP interface(Vandergrift et al. 2002; Stratford et al. 2004; Santa-Maria et al. 2006; ElGawady et al. 2007; Alcaino and Santa-Maria 2008; Santa-Maria and Alcaino 2011; Lei et al. 2016), unless the debonded length of FRP were controlled using intermediate anchorages (Holberg and Hamilton 2002; Chuang et al. 2003; Stratford et al. 2004; Lei et al. 2016).

Polyethylene Terephthalate(PET) fiber is one of the retrofitting material that has drawn a significant attention as an unique alternative to typical FRP due to its ductile behaviour, but not compromising the other advantages of FRP. Rahman and Ueda (Rahman and Ueda 2015, 2016) investigated in-plane shear performance of masonry walls...
strengthened with PET-FRP and CFRP sheets under monotonic lateral loading. The in-plane shear strength of the strengthened wall was considerably improved by using either of the FRPs, but PET-FRP had a better seismic performance than CFRP as it showed a better ductile behaviour especially in postpeak region.

In this study, PET FRP was used to strengthen unconfined and confined masonry walls which were tested under cyclic lateral loading. The purpose is to learn more about the seismic performance of masonry walls strengthened with PET FRP. Ultimate load bearing capacity, deformability and energy dissipation capacity were observed during the experiment.

**EXPERIMENTAL PROGRAM**

**Test specimens**

In this experimental study, a total of 4 clay brick masonry walls with nominal dimensions of 2115mm×1750mm×240mm were constructed. All of the walls were fabricated with bricks having a dimension of 240mm×115mm×48mm, and 10mm thick mortar. Details of the walls are shown in Figure 1 and measured material properties are listed in Table 1.

Two walls (W-1 and LW-1) were unconfined walls and the other two (W-2 and LW-2) were confined with reinforced concrete constructional columns at two ends. The constructional columns of W-2 and LW-2 were cast after the masonry wall and toothed edges were left on each side of the wall panel at the interface. The height of toothed edges was provided equal to the thickness of five brick course (290mm), and the toothing length was taken as 60mm. Columns were of rectangular cross-section with dimension of 240×125mm (the short side parallel to the plane of the wall) with the longitudinal reinforcement consisted of four 8mm diameter steel bars and 6mm diameter stirrups spaced at 150mm.

Two specimens (W-1 and W-2) were reference walls, and the other two (LW-1 and LW-2) were strengthened with 0.841mm thick, 300mm wide PET FRP sheet with unidirectional fiber. The PET FRP sheet was applied on both sides of the specimens in two different layouts as shown in Figure 1 (c) and (d). LW-1 was strengthened with PET FRP placed along two diagonals and vertical edges, and vertical strips were laid over the ends of diagonal strips to keep them in place. LW-2 was strengthened with PET FRP only placed along two diagonals with end anchorage system consisting of a 350mm×30mm×10mm steel plate and two expansion bolts placed at two ends of the steel plate.

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![Figure 1 Specimens and test setup](image-url)
Test setup

A combination of vertical compression and in-plane shear load was applied to each specimen as shown in Figure 1(e). A pre-compression of 304.6kN which was equivalent to a uniform pressure of 0.6MPa was applied on the top of the wall through two hydraulic jacks to simulate the load of structural component coming on the wall from the above in the real structure. The cyclic lateral load was then applied following a specified load-control regime. Before yielding, the applied load was increased gradually with increments of 30kN. Once yielding can be observed from hysteretic response, the displacement-control regime was adopted. At each load level, one cycle was used for each level of load controlled loading, and three full cycles were used for each displacement controlled loading. When the lateral load dropped to less than 85% of the peak load, the specimen would be considered as failed.

TEST RESULTS AND DISCUSSION

Observation of failure

W-1: At the second load-control stage (±60kN), flexural cracks appeared at the foot of the wall. After the third load-control stage (±90kN), the lateral load was applied under displacement control, and the displacement increment was 2mm starting from ±2mm. During the first ±10mm cycle, the lateral load in both directions reached the peak value, and one major diagonal crack appeared along each diagonal direction. During the second and the third ±10mm cycles, the crack width increased and the lateral load decreased. During the first ±12mm cycle, failure occurred. The cracking pattern was presented in Figure 2(a) and (b).

W-2: After the cycle of ±150kN, the displacement reached ±3.5mm and flexural cracks had already appeared at the wall toe and then the displacement controlled loading was adopted. During the second displacement control stage (±6mm), several horizontal cracks appeared on confining concrete columns. After 3 cycles of ±6mm, the displacement increment was taken as 3mm. During the first cycle of ±9mm, diagonal cracks appeared at the central area and the foot of the wall. During the first cycle of ±12mm, the lateral load in both directions reached the peak amount, and major diagonal cracks appeared along each diagonal direction. During the first ±18mm cycle, failure occurred. As shown in Figure 2(d), the cracking pattern of W-2 changed from large diagonal cracks of W-1 (as shown in Figure 2(b)) to a network of several diagonal cracks of smaller width, and dozens of flexural cracks were distributed along constructional columns.

LW-1: After the cycle of ±120kN, the lateral load was applied under displacement control, and the displacement increment was taken as 2mm starting from ±2mm. During the cycles of ±4mm, flexural cracks had already appeared at the wall toe. During the cycles of ±10mm, several diagonal cracks appeared along each diagonal
Lateral load in both directions reached the peak value. During the cycles of ±16mm ~ ±20mm, debonding of PET FRP was observed accompanied by crushed bricks falling off, and vertical cracks appeared at two ends of the wall. When the displacement reached 22mm, failure occurred and crushed bricks at two ends of the wall started to fall off. As shown in Figure 3, diagonal cracks were distributed along edges of PET FRP strips, and rupture of PET FRP strips did not happen.

LW-2: During the cycle of ±90kN, the pull displacement was much larger than the push displacement, and yielding had already occurred in the pull direction. Construction flaws in the foundation anchorage of longitudinal reinforcement in confining columns were suspected to exist, which may cause this kind of asymmetry. After the cycle of ±90kN, the displacement controlled loading was adopted. The first loading stage is 1.5mm in pushing and 3mm in pulling, and the second stage is 3mm in pushing and 4.5mm in pulling. Starting with the third stage, which is 6mm in both pushing and pulling, the increment was fixed to 3mm. During the early stages of displacement controlled loading, flexural cracks had already appeared at the wall toe. Through the cycles of ±9mm, diagonal cracks appeared along each diagonal direction. During the first cycle of ±12mm, the lateral load in pushing direction reached the peak value. During the first cycle of ±18mm, the lateral load in pulling direction reached the peak amount. During the first ±21mm cycle, failure occurred. The cracking pattern of LW-2 shown in Figure 4 was quite similar to LW-1, debonding of PET FRP was observed in some areas accompanied by crushed bricks falling off. Furthermore, a few flexural cracks were distributed along constructional columns.

Hysteretic behaviour

Figure 5 shows the hysteretic behaviour of the specimens. Before diagonal cracks appeared, all specimens displayed narrow hysteretic loops. After diagonal cracks appeared, relatively larger loops were observed. Pinching phenomenon was more obvious in hysteretic loops of LW-2 than LW-1, which may be caused by suspected construction flaws in LW-2, indicating that sliding was more serious in LW-2. Therefore, applying PET FRP along two vertical edges can be considered as an alternative to constructional columns.
Load-displacement response and stiffness

In Figure 6(a), the specimens’ envelope curves for the first cycle at each level were presented. For each specimen, the experimental value for the “yield displacement” can be obtained from intersection point of the horizontal line at the peak load and the straight line from the origin passing through the point at 75% of the peak load as shown in Figure 6(b) (Park et al. 1982), in which V is lateral load, δ is lateral displacement, V_{max} is peak load, V_{yielding} and δ_{yielding} are load and displacement at yielding respectively, and test results were summarized in Table 2.

![Figure 6 load-displacement response: (a) envelope curves; (b) calculation method of yielding point (Park et al. 1982); (c) stiffness.](image)

**Table 2 loads and displacements at yielding, peak strength and failure**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Loading direction</th>
<th>Yielding Load (kN)</th>
<th>Displacement (mm)</th>
<th>Peak strength Load (kN)</th>
<th>Displacement (mm)</th>
<th>Failure Load (kN)</th>
<th>Displacement (mm)</th>
<th>Ductility factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-1</td>
<td>+</td>
<td>250.64</td>
<td>8.48</td>
<td>277.56</td>
<td>9.68</td>
<td>235.93</td>
<td>10.02</td>
<td>1.18</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>275.15</td>
<td>7.14</td>
<td>315.65</td>
<td>9.82</td>
<td>268.30</td>
<td>10.72</td>
<td>1.50</td>
</tr>
<tr>
<td>W-2</td>
<td>+</td>
<td>287.88</td>
<td>7.31</td>
<td>327.90</td>
<td>11.90</td>
<td>278.72</td>
<td>13.73</td>
<td>1.88</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>310.68</td>
<td>8.94</td>
<td>345.50</td>
<td>10.80</td>
<td>293.68</td>
<td>13.13</td>
<td>1.46</td>
</tr>
<tr>
<td>LW-1</td>
<td>+</td>
<td>469.40</td>
<td>9.06</td>
<td>544.87</td>
<td>13.80</td>
<td>463.14</td>
<td>20.15</td>
<td>2.22</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>488.55</td>
<td>7.62</td>
<td>584.55</td>
<td>14.18</td>
<td>496.87</td>
<td>18.57</td>
<td>2.44</td>
</tr>
<tr>
<td>LW-2</td>
<td>+</td>
<td>421.06</td>
<td>7.03</td>
<td>495.51</td>
<td>11.92</td>
<td>421.18</td>
<td>16.71</td>
<td>2.38</td>
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<tr>
<td></td>
<td>-</td>
<td>454.55</td>
<td>11.66</td>
<td>527.17</td>
<td>18.00</td>
<td>448.10</td>
<td>20.72</td>
<td>1.78</td>
</tr>
</tbody>
</table>

Note: + represents the pushing direction, - represents the pulling direction. Failure occurred when the lateral load dropped to 85% of the peak load. Ductility factor is the ratio between the displacement at failure and the displacement at yielding.

Two control walls had similar load-displacement response as shown in Figure 6(a) and Table 2, but the peak strength, deformability and ductility factor of W-2 had a 16%, 29% and 25% increase respectively compared with W-1. Comparison between control walls and strengthened walls shows that the technique of strengthening with PET FRP sheets appeared to be significantly effective. The peak strength, displacement at failure and ductility factor of LW-1 were 90%, 87% and 74% higher than that W-1 respectively. The peak strength, displacement at failure and ductility factor of LW-2 were 52%, 40% and 24% higher than that of W-2 respectively.

The stiffness of each specimen is displayed in Figure 6(c). The stiffness of both control walls were almost the same at early loading stage, and it decreased rapidly before the lateral displacement reached 3mm. However, a sudden decrease of stiffness after the lateral displacement reached 10mm was observed in W-1. The stiffness of the strengthened unconfined wall LW-1 was always higher than the unconfined control wall W-1, and the decreasing of stiffness clearly slowed down after the lateral displacement reached 2mm. The stiffness of the strengthened confined wall LW-2 in two directions decreased asymmetrically. In the pushing direction, the stiffness of LW-2 was almost the same as the stiffness of LW-1 and higher than the stiffness of the control specimen W-2. In the pulling direction, the stiffness of LW-2 was lower than the stiffness of W-2 initially, but gradually became higher. It can be concluded that PET FRP sheets were effective in enhancing the stiffness of masonry wall, especially after yielding.
Energy dissipation

High-energy dissipation is a desirable property. The cumulated energy dissipation of the specimens was calculated (Figure 7(a)). As shown in Figure 7(a), the strengthened specimens dissipated higher energy compared to the control walls, as LW-1 dissipated 406% higher energy than W-1, and LW-2 dissipated 145% higher energy than W-2.

The equivalent damping coefficient ($\zeta$) was calculated as $\zeta = \frac{A_{ABC+CDA}}{2\pi A_{OBE+ODF}}$, where $A_{ABC+CDA}$ is the energy dissipated in a hysteretic loop and $A_{OBE+ODF}$ is the strain energy measured at the peak force of the same cycle, i.e., the sum of the areas under the triangles in Figure 7(b). Figure 7(c) showed the calculated equivalent damping coefficient as a function of lateral displacement for all the test specimens. Before diagonal cracks appeared, the equivalent damping coefficient of all specimens was slightly decreasing as the lateral displacement increased. After diagonal cracks appeared, the coefficient of W-1 and W-2 increased drastically while the coefficient of the strengthened walls increased much more slowly.

![Figure 7 energy dissipation analysis: (a) cumulated energy; (b) definition of damping; (c) damping coefficient](image)

**DESIGN EQUATION**

The total shear load carried by the strengthened specimen was conventionally split into two components: the shear load carried by an equivalent unreinforced masonry specimen and the increase in shear capacity due to strengthening (Stratford et al. 2004). Based on truss mechanism (ElGawady et al. 2007), a simple design equation for diagonally strengthened walls was proposed as the following form:

$$V_{\text{max}} = V_{\text{m}} + \alpha \rho h f_{\text{PET}} t L$$  \hspace{1cm} (9)

Where $V_{\text{m}}=$shear strength of unreinforced wall according to the reference walls; $\alpha=$coefficient of efficiency (based on numerical fitting according to experimental results, which is suggested to be taken as 0.38); $\rho=$reinforcement ratio of FRP in the horizontal direction; $f_{\text{PET}}=$tensile strength of the PET sheet; $t=$thickness of wall; $L=$length of wall.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$V_{\text{max}}$</th>
<th>$V_{\text{exp}}$</th>
<th>$V_{\text{max}}/V_{\text{exp}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>LW-1</td>
<td>559.7</td>
<td>564.7</td>
<td>0.99</td>
</tr>
<tr>
<td>LW-2</td>
<td>599.8</td>
<td>511.3</td>
<td>1.17</td>
</tr>
<tr>
<td>PSD1 (specimen mentioned in Rahman and Ueda 2016)</td>
<td>103.2</td>
<td>114.0</td>
<td>0.90</td>
</tr>
<tr>
<td>PSD2 (specimen mentioned in Rahman and Ueda 2016)</td>
<td>103.2</td>
<td>101.0</td>
<td>1.02</td>
</tr>
</tbody>
</table>

**CONCLUSION**

The contribution of two different configurations of PET FRP reinforcement to the in-plane shear response of unconfined and confined clay brick walls was experimentally investigated. The test results showed that the peak strength, deformability, energy dissipation capacity and ductility factor of the strengthened specimens were significantly improved, the stiffness was enhanced and the equivalent damping coefficient was stabilized attributed to the strengthening. A simple design equation for diagonally strengthened walls was proposed for future application.

Considering the cases in this paper, the strengthened unconfined masonry wall performed better than the strengthened confined wall, since LW-1 demonstrated higher shear capacity, higher deformation capacity, higher energy dissipation capacity and larger ductility. PET FRP strengthening method was more suitable for unconfined wall. Therefore, applying PET FRP along two vertical edges over the ends of PET sheets placed along two
diagonals was recommended, not only because it was proved to be effective, but also because it was easy to apply and the application can avoid construction flaws like those suspected in the confining columns of LW-2.

Further work still needs to be planned. For example, failure mechanism requires deeper understanding, design equation needs more calibration, and single-side strengthening needs to be investigated as it is often not practicable to apply the reinforcement to both sides.

ACKNOWLEDGEMENT

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REFERENCES


NUMERICAL ANALYSIS OF RC BEAMS STRENGTHENED WITH PRESTRESSED CFRP LAMINATES

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ABSTRACT

The paper presents the results of numerical analysis of reinforced concrete beams strengthened with pre-tensioned CFRP laminates verified by the experimental results. The aim of the tests was to analyse the effectiveness of the active strengthening in relation to the general passive methods of strengthening RC members in flexure and to show the influence of the loading history before strengthening on the strengthening efficiency. A two-dimensional model based on the nonlinear characteristics of component materials (cracking of concrete, elastic-plastic model of steel reinforcement, bond-slip CFRP-to-concrete model) was proposed in the paper. Numerical simulations performed for the calibrated mechanical model showed a very good agreement with experimental results. Numerical analysis was broadened to include elements of non-strengthened RC members and the strengthened ones with non-pretensioned CFRP laminates as well. The analysis confirmed a suitability of the model to predict a behaviour of RC members strengthened in flexure with the CFRP materials in the whole range of loading.

KEYWORDS

RC beams, strengthening, pretensioning, load capacity, deformability, FEM model, numerical simulations.

INTRODUCTION

Flexural strengthening of RC structures with externally bonded (EB) carbon fiber reinforced polymer (CFRP) laminates has been already widely used in engineering practice. However, this technique is effective in the case of slightly preloaded RC members before strengthening. If the structure has been cracked and highly deformed under initial extensive preloading, strengthening with non-pretensioned (passive) composites is not significantly effective. To fulfil this efficiency and to improve the serviceability state (by crack and deflection reduction) application of pretensioned CFRP laminates is recommended. Although preloading is one of the most important parameters to be accounted in the design of strengthening existing RC structures, this problem has been investigated very rarely (Wen-Wei Wang and Guo Li, 2004).

An experimental program consisting of three series of RC beams with variations in the longitudinal steel reinforcement ratio, concrete strength, preloading level before strengthening, and adhesion between the CFRP laminates was performed in Lodz University of Technology. A practical and unique aspect of this program focused on an analysis of the influence of preloading on the strengthening efficiency with pretensioned CFRP laminates. Results of the tests were used for verification of the proposed numerical model. The main aim of this paper is to develop a computational model based on displacement control finite element method using TNO DIANA software and to verify numerical simulations with the experimental results in the whole range of loading.

EXPERIMENTAL TESTS

The experimental program consisted of three series beams (A, B, and C), which contained six 500x220mm RC beams in total. The beams were strengthened under two preloading levels. The lower preloading level was equal the self-weight of the beams only (corresponding to 25% and 14% of the yield strength of non-strengthened beam) in series A and C, respectively (the difference was caused by each series having a different steel reinforcement ratio). The higher preloading level, 76% of the yield strength of the non-strengthened member, was selected to approach the elastic limit of the unstrengthened beam behaviour. To reflect the variability seen in existing structures, the longitudinal reinforcement ratio of the test beams was varied by using two different bar diameters: 12 and 16 mm. Series A and B together included 4 beams reinforced with four 12-mm-diameter bars in tension. Series C contained 2 beams reinforced with four 16-mm-diameter bars. The shear reinforcement consisted of 8-
mm-diameter steel stirrups with a 150-mm spacing. The beams were cast on three different dates with commercially-supplied Class C30/37 concrete. The members were tested in 6-point loading over a 6000 mm simple span (Figure 1). Most of the beams were strengthened with prestressed CFRP laminates of 100 x 1.2 mm ($E_f = 173$ GPa) bonded to the concrete with epoxy adhesive. Two of them were strengthened without any adhesive between the laminates and the concrete; the laminates behaved like an external bowstring attached to the beam only at the anchorage plates. An “a” index in the beam identification indicates the presence of adhesive, while an “e” index indicates the initial external preloading. A summary of all the investigated parameters and main test results are shown in Table 1. More details of the experimental tests were published in Kotynia et al. (2013). The most common failure mode, RC beams strengthened with the CFRP laminates bonded along their entire length (B12-asp, B12-asp-e, B16-asp, B16-asp-e, and B12-a), was an intermediate crack-induced (IC) debonding of the CFRP laminate initiating at one of the middle loading points and extending toward the near support. A secondary failure, occurring after IC debonding, was the CFRP sliding from under the anchorage plate.

![Figure 1 Steel reinforcement, strengthening configuration and test set-up](image)

Table 1 Summary of tested members and essential test results

<table>
<thead>
<tr>
<th>Series</th>
<th>Beam</th>
<th>Tensile reinforcement</th>
<th>Initial preloading; ($2F_p/2F_u 100%$), [%]</th>
<th>$2F_p$ [kN]</th>
<th>$\varepsilon_f,p$ [%]</th>
<th>$\sigma_f,p$ [MPa]</th>
<th>$\varepsilon_f,test$ [%]</th>
<th>$\eta_f$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>B12-a</td>
<td>#12</td>
<td>Self-weigh; (25)</td>
<td>53.0</td>
<td>5.20</td>
<td>900 (0.32 $f_u$)</td>
<td>9.30</td>
<td>86</td>
</tr>
<tr>
<td></td>
<td>B12</td>
<td>#12</td>
<td>Self-weigh; (25)</td>
<td>47.3</td>
<td>4.60</td>
<td>796 (0.28 $f_u$)</td>
<td>6.90</td>
<td>68</td>
</tr>
<tr>
<td>B</td>
<td>B12-a-e</td>
<td>#12</td>
<td>Self-weigh + external preloading; (76)</td>
<td>49.0</td>
<td>4.75</td>
<td>822 (0.29 $f_u$)</td>
<td>6.85</td>
<td>69</td>
</tr>
<tr>
<td></td>
<td>B12-e</td>
<td>#12</td>
<td>Self-weigh + external preloading; (76)</td>
<td>45.5</td>
<td>4.40</td>
<td>762 (0.27 $f_u$)</td>
<td>5.00</td>
<td>56</td>
</tr>
<tr>
<td></td>
<td>B16-a</td>
<td>#16</td>
<td>Self-weigh; (14)</td>
<td>74.4</td>
<td>4.80</td>
<td>831 (0.29 $f_u$)</td>
<td>8.00</td>
<td>76</td>
</tr>
<tr>
<td>C</td>
<td>B16-a-e</td>
<td>#16</td>
<td>Self-weigh + external preloading; (76)</td>
<td>72.0</td>
<td>4.85</td>
<td>840 (0.29 $f_u$)</td>
<td>7.15</td>
<td>71</td>
</tr>
</tbody>
</table>

$2F_p$ – Initial preloading; $2F_u$ – Failure load
$2F_p/2F_u 100\%$- Initial preloading level in comparison to the yield strength of non-strengthened beam;
$\varepsilon_f,p$, $\sigma_f,p$ - Pretensioning strain and stress in the CFRP laminate during strengthening; $\varepsilon_f,test$ - Maximal CFRP strain registered in the test
$\varepsilon_f,test$ - Total CFRP strain, $\varepsilon_f,test = \varepsilon_f + \varepsilon_f,\text{test}$; $\varepsilon_u$ - Ultimate CFRP strain; $\eta_f$ - Utilisation of the tensile CFRP strain, $\eta_f = \varepsilon_f,\text{test} / \varepsilon_u 100\%$

**NUMERICAL ANALYSIS**

Finite element model for beams

Numerical analysis of strengthened beams with prestressed CFRP laminates was performed using DIANA finite element code. The specimens were modelled in 2D (plane stress) and the topology of the finite element mesh adopted for calculation is presented in Figure [Error! Reference source not found.]. The finite element mesh of concrete matrix consists of quadrilateral eight-node isoparametric plane stress elements with shape functions described by second order polynomials.

![Figure 2 Finite element model for RC beams; a) mesh topology, b) configuration of steel reinforcement](image)
Maximum dimension of each finite element does not exceed 15 mm. The bottom steel reinforcement was modelled using three-node truss elements connected to the concrete matrix by special interface elements. This type of connection is able to model the relative displacements between the concrete matrix and reinforcing bars in the direction tangential to the reinforcement. In order to model the bond-slip behaviour between the composite laminate and concrete surface six-node interface elements with zero thickness was used.

**Numerical model of loadings and constitutive relationships for materials**

Four loading schemes were considered in numerical calculations. These loading schemes include: the dead weight, shrinkage of concrete, prestressing applied to laminates, external preloading and additional loading in the form of concentrated forces. The loading conditions were applied in sequences that follow the experimental loading program described in the previous section.

The dead weight was modelled as the mass forces applied to each node of finite element mesh. The concentrated load \( F \) was replaced by the local, uniformly distributed load over the width of 0.1 m. The evolution of shrinkage strain \( \varepsilon_{sh}(t) \) was modelled according to Eurocode 2 (2004). The shrinkage strain was added to mechanical strains of concrete in ten calculation steps to reflect the development of this strain in the time period from removing the formworks to performing the experiment. The prestress was modelled as the uniformly distributed load \( \sigma_p \) applied to the ends of CFRP laminate. The value of \( \sigma_p \) was calibrated in such a way that after immediate losses due to slip in the anchors and deformations of the specimen the value of initial strain in laminate \( \varepsilon_{p} \) obtained in calculations was equal to the strain measured in the tests (see Table 1).

The constitutive model for concrete adopted in the current analysis are based on the concept of smeared crack and is formulated in the total strains following the concept proposed in (Vecchio (1989), Vecchio (1990), Maine edit. (2014)). The fixed crack approach was used in this study. Before cracking the stress-strain relationships are evaluated in the directions of principal strains. After cracking the local directions are fixed and the stress-strain relationships are evaluated in the coordinate system determined by the first crack. Additionally, in the direction tangential to the crack the shear stiffness is reduced (Kirchhoff modulus is multiplied by shear retention factor \( \beta \times 1.0 \)). Uniaxial stress-strain relationships for concrete in compression and tension were taken after Feenstra (1993) and Cornelissen et al. (1986), respectively and they are presented in Figure 3a.

The mesh objectivity of the numerical solution was obtained by maintaining the constant fracture energy in tension \( G_f \) and compression \( G_c \) during cracking and crushing process – see Figure 3a. The crack bandwidth \( h \) was taken equal to square root of a finite element area. The main adopted material parameters for concrete are following: \( E_c = 23.7-26.7 \text{ GPa; } \nu_c = 0.2; f_c = 28.7-51.0 \text{ MPa; } f_t = 1.9-4.1 \text{ MPa; } G_f = 90 \text{ N/m; } G_c = 10^8 \text{ N/m;} \) The elastic-plastic with kinematic hardening model was used for the steel reinforcement. The material parameters that describe uniaxial behaviour of steel reinforcement (i.e. elastic modulus, yield stress, tensile strength) are determined from experimental tests by Kotyna et al. (2013).

**a)**

![Figure 3 a) uniaxial constitutive model for concrete](image)

**b)**

![Figure 3 b) traction-slip law between concrete and laminate](image)

In the case of beams strengthened in flexure with CFRP laminates the normal forces between a laminate and concrete are usually small. Due to this fact the physical relationships between tractions and relative displacements in the concrete-laminate interface are uncoupled, i.e. traction \( t_n \) in the normal direction is a function of the relative normal displacement \( \vec{u}_n \) and shear traction \( t_s \) depends only on shear relative displacement \( \vec{u}_s \). In the present analysis the normal traction is the linear function of the relative normal displacement: \( t_n = K_n \vec{u}_n \), where \( K_n \) is the normal stiffness. In the shear direction the bond-slip behaviour is described by the formula \( t_s = g_s(\vec{u}_s) \) according to Lu et al. (2005). For beams strengthened without the adhesion between concrete surface and laminate (specimens B12, B12-e) the nonlinear elastic friction model (Maine edit. (2014)) was used. The relationships...
between the tractions and the relative displacements are described by Coulomb law with two materials parameters – friction coefficient $\mu$ and cohesion $c$. For this simplified model the loading and unloading process follows the same curve. In the current study the following material properties were adopted: $K_e = 0.07\text{GPa/m}$, $\mu = 0.2$, $c = 0.15\text{MPa}$ in the case of beam B12 and $K_e = 0.05\text{GPa/m}$, $\mu = 0.2$, $c = 0.12\text{MPa}$ for element B12-e. The small value of cohesion was assumed in order to stabilize calculations.

An incremental iterative procedure was employed in order to obtain the solution for analysed structures and loading programs. The computational process was controlled by increments of external loads and prestress. For load levels close to the failure load the force controlled procedure was changed to the arc-length method. For each load increment the equilibrium between internal and external forces was calculated using Newton-Raphson procedure. The force and displacement norms were used as convergence criteria.

Test procedure contained several stages, which were precisely reflected in the numerical simulations using a phased analysis. The following phases were taken into consideration: phase I – application of the shrinkage strain to the concrete matrix; activation of concrete and steel elements; phase II – preloading of the active elements with a dead load and external load; phase III – activation of CFRP elements; CFRP pretensioning; phase IV – activation of the first new interface elements; application of the prestressing load at the CFRP ends with gradual force decreasing and transferring of prestressing by the anchors to the beam; phase IVa – deactivation of temporary supports (in the beams with external preloading only); phase V – further loading of the beam until failure.

**Comparison of experiments and numerical results**

The main goal of comparative analysis is a verification of the adopted numerical model. In the analysis all quantities measured during experiments are taken into account, i.e. vertical displacements, mean compressive and tensile strains in concrete averaged at the zone of constant curvature and mean strains of composite averaged at the same zone. Moreover, the analysis includes a comparison with calculated reference beams, i.e. non-strengthened beams and beams strengthened with passive laminates. Comparison of calculated and test load-displacement curves is shown in Figure 4. Proposed model provides the opportunity to analyze the influence of prestressing and preloading levels on the strengthening effectiveness in ULS and SLS.
The experimental and numerical results are in a very good agreement for the whole loading range and all experimental stages. It should be emphasized that numerical model is able to reproduce a reduction of stiffness due to cracking at the first stage of experiment (preloading stage), then reflects the stiffness recovery due to prestressing and finally again decreasing of stiffness as a result of crack development and slips between laminate and concrete. A good accuracy in simulations of the beams behaviour was achieved both for specimens strengthened under dead load and for the beams preloaded up to 76% of the yield strength of the nonstrengthened beam. Figures 5 and 6 present comparison of the test and predicted mean strains in the CFRP laminates and concrete (measured at the distance of 35 mm from the top and bottom surface of the specimens). The predicted CFRP strain is satisfying both for beams with and without adhesive layer between the anchor plates. The model correctly reproduces changes of the CFRP stain gradients due to crack development and the steel yielding. The comparison of numerical and experimental concrete strains (mean at the constant curvature distance, Figure 6) confirms that model is correctly calibrated and it is capable to predict the concrete compressive and tensile strain.

Numerical results verified by experimental ones (Figures 4-6) indicate that the numerical model is valuable for further parametric studies of the RC members strengthened in flexure with pretensioned FRP reinforcement. The

Figure 5 Calculated and test charts of load – CFRP strain

Figure 6 Calculated and test charts of load – concrete strain
prestress losses determined in the test with decrease in the CFRP strain based on the strain gauges, confirmed the predicted ones.

Figure 7 describes the numerical failure mechanism in the laminate-concrete interface. It shows developed debonding process between laminate and concrete surface that initiates from the middle loading point and propagates towards the support. The same failure mode was obtained on the tested beams strengthened with externally bonded CFRP laminates. Thus it can be concluded that failure mechanism observed in experiments is correctly reproduced by numerical simulation.

![Figure 7 Slip between concrete and CFRP laminate: a) specimen B12-a, b) specimen B16-a-e, x parameter is a distance from the support](image)

**SUMMARY AND CONCLUSIONS**

On the basis of a comparative analysis of the test and numerical results the following conclusions can be drawn:

- Flexural strengthening with prestressed CFRP is an efficient technique of strengthening RC members.
- Despite the preload levels in some cases exceeding the serviceability limit states prior to strengthening, the application of prestressed CFRP laminates resulted in a significant reduction of deflections and strains due to subsequently applied loads and led to a recovery of beam stiffness to a non-preloaded value.
- The developed model was used to simulate the experimental RC beams strengthened with pretensioned CFRP laminates. The model is capable to simulate the nonlinear behavior of the constituent materials.
- Comparison of experimental and computational results confirms a good predictive performance of the MES model in terms of the flexural response of RC beams strengthened with pretensioned CFRP laminates over the range covering pre-loading state, strengthening process, up to failure. It is useful for analysis of the crack propagation.
- The proposed FE model successfully predicts enhancing effects in the SLS and ULS and it can be used for parametric studies considering variable investigated parameters: elasticity modulus and cross section of FRP laminates, steel and FRP reinforcement ration, concrete strength, FRP pretensioning levels and preloading levels.

**REFERENCES**


EXPERIMENTAL STUDY OF INTERMEDIATE CRACK DEBONDING FAILURE IN FRP-STRENGTHENED CONCRETE BEAMS

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ABSTRACT
The intermediate crack (IC) induced debonding failure in externally bonded FRP-strengthened concrete beams is a main premature failure type. This failure is due to the high interfacial stresses at the locations of shear/flexural cracks. This study presents an experimental program to investigate the behavior of IC debonding failure and bond-slip relation between the FRP plates and concrete in FRP-strengthened beams. A notch was made by saw cut in concrete beam to represent a major flexural/shear crack that triggers IC debonding. To study the sensitivity of the bond-slip behavior to the location of the major flexural/shear crack through the beam span, the notch located at different locations with the beam span in different specimens. In all experimental cases, it was observed that the IC debonding initiated at the mouth of a diagonal crack close to the notch or the flexural/shear crack. The experimental results and the related numerical analysis indicate that the bond behavior of the interface is not sensitive to the location of the major flexural/shear crack.

KEYWORDS
FRP, IC debonding, concrete beam, bond-slip behavior, concrete/FRP interface.

INTRODUCTION
Intermediate crack (IC) induced debonding failure in reinforced concrete beams strengthened with Fiber Reinforced Polymer (FRP) composite materials starts at the mouth of flexural/shear cracks within the shear span and propagates towards the FRP plate termination. One of the common methods to determine the FRP/concrete bond behavior is the single shear pullout test (e.g., Chajes et al. 1996, Taljsten 1997, Leung and Tung 2006, Mazzotti et al. 2008, Mohammadi and Wan 2015). In this method, the in-plane shear stress is applied to the FRP/concrete interface by applying a uniaxial tension load in the plane of the FRP, typically in the strong or longitudinal direction of the FRP fiber orientation. The tensile strain gradient in the FRP (representing the shear strain along the interface) is recorded and is used to determine the bond-slip relation. The obtained bond-slip relation is often used for nonlinear fracture mechanics or cohesive crack models in numerical analyses to predict FRP debonding from the concrete substrate.

IC debonding failure in FRP-strengthened RC beams is due to the development of high interfacial stresses at locations of flexure/shear cracks in concrete. Since the FRP/concrete interface in strengthened beams is subjected to both flexural and shear loadings, the use of pull-out test results, in which the interface is subjected to pure shear stress, to predict the behavior of FRP-strengthened concrete beams may not represent the in situ phenomenon. Therefore, some studies have used beam tests to study the debonding behavior in FRP-strengthened RC beams (e.g., Fukuzawa et al. 1997, Benjeddou et al. 2007, Gartner et al. 2011). In this study, the beam test was used to investigate the IC debonding failure mechanisms using notched FRP-strengthened concrete beams. In order to simulate the major flexural/shear crack that triggers IC debonding failure, a half-depth notch was cut in the beam specimen. To investigate the sensitivity of the bond-slip behavior to the location of the major flexural/shear crack, the notch position in specimens varied from the midspan to the FRP plate end.
EXPERIMENTAL PROGRAM

The 28-day compressive strength of the concrete in the beam specimens was 30.4 MPa. The tensile strength of the beam specimens was determined from split cylinder tests to be 3.2 MPa. The preformed CFRP laminate strip used for strengthening was 51 mm wide and 1.5 mm thick. The manufacturer reported tensile modulus, ultimate tensile strength, and Poisson’s ratio were 155 GPa, 2800 MPa, and 0.25, respectively. The epoxy adhesive for bonding CFRP plate to concrete was approximately 1 mm thick. Its manufacturer reported tensile modulus, ultimate tensile strength, and Poisson’s ratio were 4.48 GPa, 24.8 MPa, 0.3, respectively.

Three CFRP-strengthened beams without notch were used as control specimens to investigate the behavior of the strengthened beams without cracks. Six FRP-strengthened beam specimens with a half-height notch at mid-span, as shown in Figure 1a, were tested to observe the IC debonding failure when the major flexural crack is at the mid-span of the beam. In order to investigate the sensitivity of the bond-slip behavior to the location of the major flexural/shear crack, the notch position in specimens was moved forward to the FRP plate end every 51 mm from the mid-span to the support (Figure 1b). Three specimens were tested for each position of the notch. The width of the notch was 5 mm and its height was half of the beam height (76 mm).

The test was conducted under displacement control. During the test, the applied load, the axial strains in FRP plate at different locations along the FRP plate, and the vertical deflection at the top of the notch were recorded. In the following, the experimental observations for each group of specimens are described in detail.

RESULTS AND DISCUSSIONS

Control Specimens CC-1, CC-2, and CC-3

The specimens CC-1, CC-2, and CC-3 were CFRP-strengthened beams without notch. Failure types of all three specimens were IC debonding failure and the failure processes included:

- A major vertical flexural crack initiated very close to the mid-span of the concrete beam (Figure 2a);
- A diagonal flexural/shear crack started close to the first flexural crack, about 25 to 40 mm from the first one (Figure 2b);
- FRP debonding along the FRP/concrete interface initiated at the mouth of the diagonal crack and propagated toward the support (Figure 2c);
- Sudden and essentially instantaneous failure steps included: the diagonal crack merging with the major flexural crack, the merged crack propagating to the loading point, and FRP debonding continuing to the end of the FRP plate (Figure 2d).
Specimens CMC0-1 to CMC0-6 were the strengthened beams with notch at the mid-span. The IC debonding failure processes in this group of specimens include:

- Flexural cracking initiated at the top of the notch (Figure 3a);
- A diagonal flexural/shear crack started close to the notch, about 25 mm from the notch (Figure 3b) at one or other side of the notch, or occasionally both sides;
- FRP debonding along the FRP/concrete interface initiated at the mouth of the diagonal crack and propagated toward the support (Figure 3c);
- Sudden and essentially instantaneous failure steps included: the diagonal crack merging with the notch, the crack at the top of the notch propagating to the loading point, and FRP debonding continuing to the end of the FRP plate (Figure 3d).

In all specimens, there was a wedge of concrete attached to the FRP plate (Figure 3d) that shows diagonal cracking inside the concrete. In all cases, the debonding cracking started at the mouth of the diagonal crack to either side of the notch.

By comparing Figures 2 and 3, it can be seen that the failure processes are compatible. The strengthened beams with notch at midspan followed the debonding behavior of the un-notched beams. Figure 4 compares the load vs. mid-span displacement curves for the control specimens (CC series) and those with notch at midspan (CMC0 series). Specimens CMC0 are shown by dashed lines and specimens CC are shown by solid lines. As it can be seen, specimens CC behaved stiffer before the mid-span cracking and then behaved like the specimens CMC0. It is reasonable because specimens CC did not have a notch. Once the mid-span crack initiated in the specimens CC, their behavior became very close to those of specimens CMC0, where a notch was cut at the mid-span. Therefore, specimens CMC0 could simulate the real behavior of the FRP-strengthened concrete beams (specimens CC) in their ultimate stage. This verifies the original idea to evaluate the FRP/concrete bond in real beams when this type of specimen was designed by Harries et al. (2012).
(a) Vertical flexural crack  
(b) Diagonal crack 

(c) FRP bonding initiation  
(d) Final failure 
Figure 3 IC debonding failure processes of beam CMC0-5

In order to study the sensitivity of IC debonding failure to the location of the major flexural/shear crack, the notch location was moved away from the mid-span in intervals of 51 mm. In the specimens CMC2 and CMC4 series (notches at 51 and 102 mm away from the mid-span, respectively), a vertical flexural crack formed at the location close to the mid-span at first and then the failure processes were the same as the mid-span notched beams (Figures 5a and 5b). It means that the notch triggered the IC debonding. However, in specimens CMC6 (notch at 153 mm away from the mid-span), the debonding started close to the flexural crack at the mid-span and propagated in the side without the notch (Figure 5c).

CFRP-Strengthened Beams with Notches away from Mid-span

Figure 4 Load vs. mid-span displacement curves of specimens CMC0 and CC
Table 1. Average of maximum bearing loads of beam specimens

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Number of specimens</th>
<th>Average maximum load (KN)</th>
<th>Coefficient of Variation (COV)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CC</td>
<td>3</td>
<td>20.25</td>
<td>0.018</td>
</tr>
<tr>
<td>CMC0</td>
<td>6</td>
<td>20.22</td>
<td>0.047</td>
</tr>
<tr>
<td>CMC2</td>
<td>3</td>
<td>20.57</td>
<td>0.075</td>
</tr>
<tr>
<td>CMC4</td>
<td>3</td>
<td>21.31</td>
<td>0.016</td>
</tr>
<tr>
<td>CMC6</td>
<td>3</td>
<td>20.23</td>
<td>0.021</td>
</tr>
</tbody>
</table>

It is worth to mention that the concrete wedge attached to the FRP plate was seen in all specimens due to the diagonal crack that formed close to the major flexural/shear crack or the notch. The comparison of debonding processes of specimens CMC6 and CMC0 shows that the FRP debonding in both groups of specimens followed the same behavior. Therefore, the specimens with the notch 153 mm away from the mid-span essentially behaved exactly as the specimens without any notch.

The average and the coefficient of variation (COV) of the maximum applied loads of each group of the specimens are presented in Table 1. The maximum loads of CC, CMC0 and CMC6 are almost the same. Also the failure processes of these specimens (CC, CMC0, and CMC6) followed the same behavior as described before. The FRP debonding failure of CMC6 initiated at the mouth of the diagonal crack close to the mid-span flexural crack as the specimens CC and CMC0.

The maximum loads of CMC2 and CMC4 are slightly higher (about 1.7 and 5.4% higher than that of CMC0, respectively) and CMC4 has a maximum load 3.6% larger than that of CMC2. It seems that as the major flexural/shear crack was moved away from the mid-span, but not too far (less than 153 mm in this test), the maximum flexural loading capacity increased slightly. Of course, the numbers of specimens in each group are too small to make an accurate conclusion.

**FINITE ELEMENT ANALYSIS**

The commercial software ABAQUS/standard 6.13 was used for the finite element (FE) analysis. The plane stress 4-node bilinear 2D elements (CPS4R) were used to model all materials. The behavior of the FRP and the epoxy
were simulated using a brittle cracking model. The concrete was modeled using cohesive method in extended finite element method (XFEM). The XFEM-based cohesive method uses the traction-separation law to model crack initiation and propagation along an arbitrary, solution-dependent, path since crack propagation is not tied to the element boundaries in a mesh. In the FE analysis in this study, the crack initiation criterion was the maximum principal stress criterion, in which a crack was initiated if the maximum principal tensile stress reached the tensile strength of the concrete, \( f_t \). The crack propagated perpendicular to the direction of the maximum principal tensile stress. The evolution of the crack was governed by the mode I fracture energy of concrete, \( G_F \), which represents the assumed linear tension-softening behavior of the concrete during cracking. The detail of the FE analysis can be found in Mohammadi (2014).

The numerical analysis shows that the element at the mouth of the notch is not under pure shear stress and this is the reason for the formation of the diagonal crack (Mohammadi 2014). However, the FRP/concrete interface at the mouth of the diagonal crack is subjected to pure shear stress after the initiation of the diagonal crack regardless of the location of the major flexural/shear crack. Therefore, the bond-slip behavior of the FRP/concrete interface is not sensitive to the location of the major crack.

CONCLUSIONS

Through the experimental program, it was found that the FRP debonding in IC debonding failure in FRP-strengthened beam initiates at the mouth of a diagonal crack close (about 25mm in this study) to the major flexural/shear crack. The IC debonding failure is triggered by the major flexural/shear crack in the zone with large moment/shear ratio in a FRP-strengthened concrete beam. The load capacity of the beam increased slightly when the major flexural/shear crack was moved 51 and 102 mm away from the mid-span. However, the beam with notch 153 mm away from the midspan had similar load capacity as those without notch or with notch at the midspan. The finite element analysis shows that the bond-slip behavior of the FRP/concrete interface is not sensitive to the location of the major crack.

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ABSTRACT

Use of carbon fiber reinforced polymer (CFRP) systems for strengthening concrete structures has grown considerably in recent years in Russia. The effect of preload and crack repairing factors on the debonding behavior of CFRP-strengthened reinforced concrete (CR) beams was studied in the Laboratory at Perm National Research Polytechnic University. A number of bending tests were performed to study the behavior of 22 medium-scale beams 1.27 m long having a cross section of 12 x 22 cm. Infrared thermography (IRT) was used to detect early signs of debonding. The following interesting results were obtained. Strengthening of RC beams with CFRP carried out both before and under loading provides a substantial increase in their bearing capacity. The process of CFRP debonding in the beams reinforced under loading after the appearance of first cracks and their grouting begins at strain by 4-65% lower than that of the beams reinforced before loading, and the relative area of debonding increases by 2.1-2.3 times. Grouting of cracks in beams under loading allows one to increase the limiting value of a bending moment by 45-71% compared to unreinforced beams. The CFRP debonding strain is weakly dependent on the strength of the concrete substrate. For the beams strengthened with CFRP with transverse wrapping anchorage in support sections, the FRP debonding is not in itself a failure mode.

KEYWORDS

FRP, RC beams, strengthening, IC debonding, infrared thermography, quality control, preload factor, crack repairing factor

INTRODUCTION

Various types of fibrous, woven-fabric, composite materials are widely used for structural repair and strengthening of concrete structures (Teng 2001, Teng and Chen et al. 2003). One of the common strengthening techniques is based on gluing the composite material to the concrete surface in the area on which tensile strains act. The efficiency of repair and reinforcement methods is associated with the interaction of strengthening elements with concrete surface. Some works (e.g. Zhang et al. 2006; Parikh et al. 2012; Al-Salloum) present the results of studies investigating the influence of the degree of preloading on the strength and stiffness of reinforced beams. In the present work, using the experimental data, we examine how much the interaction of composite material with concrete surface would depend on the instant of strengthening – before loading or during loading after first cracks start. We find that debonding between two materials is an important stage in the load-deformation behavior of the system “composite material-concrete”. Application of a new method that is based on infrared thermography technique allows us to successfully register debonding.

TEST PROGRAM AND PROCEDURE

Our experiments were carried out with two groups of reinforced concrete beams (B20 and B40). The scheme of a sample strengthened with steel reinforcement rods and CFRP layer is shown in Figure 1. Each of the groups was divided into 3 series with 3-5 samples: series A – control samples without CFRP layer; series B – beams reinforced prior to loading with CFRP sheet SikaWrap-230 (6 cm wide) attached to the beam with transverse wrapping anchorage by CFRP straps 20 cm wide in two support sections of the beam; series C - beams reinforced (during loading) with CFRP sheet after the appearance of first cracks and their epoxy injection repair. The basic test setup
for the four-point bending test is shown in Fig. 2a. Strain gauges were installed on steel reinforcement rods, the carbon-fiber sheet and the surface of all beams to control deformations along the beam axes.

The beams were loaded in a stepwise fashion; every loading step of 2 kN accounts for 4-6% of the fracture load. During each loading step the samples were “aged” for 5-10 minutes, which made it possible to register the crack pattern and the crack opening width. Simultaneously, they were heated and thermal pictures of the beam surface covered with a carbon fiber sheet were taken with a FLIR T620 infrared camera (e.g. Valluzzi et al. 2009; Shirazi and Karbhar 2007; Taillade et al.). Shooting parameters (power and time of heating, time of observation) were determined in advance by analyzing the numerical solution of the nonstationary heat conduction problem for the system “carbon fiber sheet – epoxy resin – concrete – debonding – concrete” (Bykov et al. 2016). The analysis shows that the best conditions for registration of debondings are achieved at the stage of beam cooling at 19th second after the start of observations (heating time 10 sec, cooling time 9 sec, heating power 932 Bt/m²). Shots were taken “through the mirror”, which ensured the safety of people and equipment at loads close to the destruction of the beam (Figure 2b).

Thermography images of the surface of the beam reinforced with the carbon fiber sheet were obtained at each loading step. The initial thermogram for each j-th step (Fig. 3a) is a two-dimensional array of differential temperature values $T_j(x, y)$ determined during the 19th and 0th seconds at points with coordinates $(x, y)$. The index $j \in [0, N]$ specifies the number of loading step; loading is absent at $j = 0$. The obtained initial thermograms were processed using an algorithm specifically designed using Matlab programs.
In the first step of the algorithm, we calculate the normalized thermograms $TN_j(x, y)$:

$$TN_j(x, y) = T_j(x, y) \frac{T_0(x', y')}{T_j(x', y')}$$

(1)

where $T_j(x, y)$ is the initial temperature difference at the $j$-th loading step at the point $(x, y)$, $T_0(x', y')$ and $T_j(x', y')$ are the initial temperature differences at the 0-th and $j$-th loading steps at the point $x', y'$ where no debonding is known to be present. Normalized thermograms for successive loading steps are given in Fig. 3b.

Next the temperature contrast $C_j(x, y)$ (Figure 3c) is determined:

$$C_j(x, y) = \frac{TN_j(x, y) - T_0(x, y)}{T_0(x, y)} \times 100\%$$

(2)

In order to make a decision on the existence of debonding at the point with the coordinates $(x, y)$, we calculate a threshold value for the temperature contrast $C^*$. For making it estimate, we determine, at each loading step, the average value $\overline{C_j}$ and the standard deviation $\sigma_j$ in those areas of the thermograms, where debonding is known to be absent. The threshold value is calculated by the formula $C^* = \overline{C_j} + 3\sigma_j$. The areas of the CRFP layer surface, where the temperature contrast value exceeds the threshold value are identified as the areas with debonding and the remaining ones as the areas free of defect. In the binary defect map shown in Fig. 3d, the defect-free areas are shown in white color and the areas of debonding in black color.

**RESULTS AND DISCUSSIONS**

The averaged data of the static test results for the beams of series A and B are summarized in Table 1, where $M_{crc}$ is the bending moment corresponding to the onset of cracking, $a_{crc}$ is the maximum crack opening width, $f_{el}$ is the elastic deflection, $M_{ult}$ is the maximum bending moment, $f_{ult}$ is the maximum deflection, $\varepsilon_{f,ult}$ is the strain of the carbon-fiber sheet at rupture. The beams of series C were tested by loading in two stages: 1) loading of unstrengthened beams until the occurrence of first visible cracks, 2) further loading of beams after the repair actions on the loaded beam. During the restoration procedure, the low-viscous epoxy material was injected into the cracks formed on the stretched surface of the beam. Then the beam was strengthened with a CFRP layer stuck to its surface with epoxy adhesive. Our experiments indicate that the debonding develops according to the cohesive scenario, i.e. the carbon fiber sheet separates together with concrete fragments. Adhesive debonding was observed in the samples, whose surface was not prepared properly before the strengthening strip was stuck.
Table 1 Results of the static test of beams

<table>
<thead>
<tr>
<th>concrete class</th>
<th>series</th>
<th>$M_{cr}$, kN·m</th>
<th>$f_c$</th>
<th>$M_{ult}$, kN·m</th>
<th>$f_{ult}$</th>
<th>$\alpha_{etc}$</th>
<th>$\varepsilon_{ult}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>B25</td>
<td>A</td>
<td>4.0</td>
<td>0.183</td>
<td>6.83</td>
<td>16.92</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>4.68</td>
<td>0.227</td>
<td>10.17</td>
<td>8.72</td>
<td>0.9</td>
<td>13530</td>
</tr>
<tr>
<td></td>
<td>C (1 step)</td>
<td>3.6</td>
<td>0.164</td>
<td>5.19</td>
<td>1.2</td>
<td>11540</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C (2 step)</td>
<td>6.86</td>
<td></td>
<td>10.29</td>
<td>10.53</td>
<td>1.4</td>
<td></td>
</tr>
<tr>
<td>B40</td>
<td>A</td>
<td>4.96</td>
<td>0.221</td>
<td>7.12</td>
<td>15.36</td>
<td>2.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>5.59</td>
<td>0.242</td>
<td>10.76</td>
<td>8.76</td>
<td>1.1</td>
<td>13420</td>
</tr>
<tr>
<td></td>
<td>C (1 step)</td>
<td>4.8</td>
<td>0.198</td>
<td>5.55</td>
<td>1.0</td>
<td>13305</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C (2 step)</td>
<td>7.19</td>
<td></td>
<td>11.0</td>
<td>10.25</td>
<td>1.1</td>
<td></td>
</tr>
</tbody>
</table>

The average bending moment – deflection curves for each of the three series of tests are shown in Figure 4. A comparison of these results shows that the reinforcement of beams with carbon fiber layer significantly improves the bearing capacity of beams. For the beams of series B and C, the maximum bending moment (a cross mark in the figure) turned out to be higher by 37-39% ($\delta M_{ult}$) and 38-49% ($\delta M_{ult}$), respectively, than that of the non-reinforced reference beams of series A. Inspection of the graphs for beams without preliminary reinforcement (series A and C) shows that the stiffness of beams at loading steps followed by the occurrence of first cracks in concrete (the areas enclosed in a circle) reduces sharply. In the beams preliminary strengthened with a carbon fiber sheet (series B), this process is partially compensated by the strengthening layer. Curve 3 clearly demonstrates the efficiency of the repair actions carried out on the loaded beam. Injection of the cracks appeared at the first loading step allows one to restore the stiffness of beams to nearly the initial state (the area enclosed by a dashed circle). Under further loading, the deformation behavior remains the same as that for the preliminary reinforced beam (curve 2). Failure in reinforced beams (series B and C) is associated with rupture of the carbon fiber layer. Our investigations indicate that the restoring procedure for loaded beams enables one to increase the value of the bending moment corresponding to the onset of secondary crack generation by 45-71% in comparison with the unreinforced beams.

![Figure 4 Bending moment-Deflection relationships: 1 –series A, 2 –series B, 3 – series C](image)

A point corresponding to the onset of cohesive debonding of carbon fiber is marked on curves 2 and 3 by a triangle. On average, the beginning of debonding corresponds to the bending moment equal to 75% of the limit value. On debonding, the beam continues to perceive the load, yet no reduction in the beam stiffness takes place. From the onset of carbon fiber layer debonding and up to the total loss of the bearing capacity, the beam has significant strength reserve ($\delta M_{ult}$, $\delta M_{ult}$). Therefore, the debonding of the CFRP layer cannot be considered to be the limit state of a beam. In the beams reinforced during loading after the appearance of first cracks and their injection, the debonding of the carbon fiber layer took place at its deformation less on average by about 25% than that of the beams reinforced in the absence of load (Table 3).

The thermogram transformation algorithm proposed in this study can be used to compute the percentage of debonding areas in different samples (Table 4). The relative area of debonding in the beams reinforced under loading (after initiation of first cracks and their injection) is on average 2.1-2.3 times greater than in the beams reinforced in the absence of loading.

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Table 3 Summary characteristics of the initiation of cohesive debonding

<table>
<thead>
<tr>
<th>Concrete class</th>
<th>Series</th>
<th>Bending moment, kNm</th>
<th>Bending moment, average, kNm</th>
<th>Crack width, mm</th>
<th>CFRP strain, με</th>
<th>CFRP strain, average, με</th>
</tr>
</thead>
<tbody>
<tr>
<td>B25</td>
<td>B</td>
<td>7.77-8.58</td>
<td>8.18</td>
<td>0.5-0.75</td>
<td>6495-8300</td>
<td>7397</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>7.62-7.92</td>
<td>7.77</td>
<td>0.5-0.7</td>
<td>5260-5780</td>
<td>5520</td>
</tr>
<tr>
<td>B40</td>
<td>B</td>
<td>7.43-7.98</td>
<td>7.71</td>
<td>0.3-0.75</td>
<td>5465-7295</td>
<td>6380</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>7.50-8.40</td>
<td>7.95</td>
<td>0.3-0.5</td>
<td>5765-6275</td>
<td>6020</td>
</tr>
</tbody>
</table>

Table 4 Relative debonding area at an average level of strain in CFRP 10300 με

<table>
<thead>
<tr>
<th>Concrete class</th>
<th>Series</th>
<th>Typical binary card of defects</th>
<th>Relative area of debonding, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>B25</td>
<td>B</td>
<td></td>
<td>9.71</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td></td>
<td>22.93</td>
</tr>
<tr>
<td>B40</td>
<td>B</td>
<td></td>
<td>14.90</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td></td>
<td>32.41</td>
</tr>
</tbody>
</table>

A comparison of the experimental strain values associated with the onset of cohesive debonding of CFRP with the theoretical results obtained by 5 commonly used methods demonstrates their low reliability (Figure 5). For instance, the values computed in compliance with Russia standards for designing reinforced concrete structure [2] exceed the experimentally registered values by 15-75% for different classes of concrete. As for the tendency for an increase in relative debonding strain with concrete strength specified in these methods, no evidence supporting it has been found in our experimental studies.

![Figure 5 Comparison of experimental and theoretical values of cohesive debonding strain: 1 – CNR-DT 200/2004; 2 – ACI 400.2R-08; 3 – Lu et al. 2007, 4 – SP 164.1325800.2014; 5 – Teng and Smith et al. 2003](image)

Our experimental studies have revealed the inconsistency between the results obtained for different series: in series B (Fig.5, circle markers), debonding in the samples of concrete class 25 is realized at CFRP strain higher by 16-19% than that of the samples of concrete class 40; in series C (Fig.5, square markers) debonding in the samples of concrete class 25 is realized at a CFRP strain lower by 10-34% than that of the samples of concrete class 40.

**CONCLUSIONS**

1. Our experimental studies have revealed the inconsistency between the results obtained for different series: in series B (Fig.5, circle markers), debonding in the samples of concrete class 25 is realized at CFRP strain
higher by 16-19% than that of the samples of concrete class 40; in series C (Fig.5, square markers) debonding in the samples of concrete class 25 is realized at a CFRP strain lower by 10-34% than that of the samples of concrete class 40.

2. It has been established that the process of CFRP debonding in the beams reinforced under loading after the appearance of first cracks and their grouting begins at strain by 4-65% lower than that of the beams reinforced before loading, and the relative area of debonding increases by 2.1-2.3 times.

3. It has been found that the onset of debonding of the CFRP corresponds to the bending moment, which is 75% of the limit value. Therefore, the debonding does not imply the onset of failure (a limit state of) CFRP beams with transverse wrapping anchorage in support sections. This makes it possible to use the composite breaking strain as a limit state of such a beam (according to the first basic group of limit states).

ACKNOWLEDGMENTS

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Taillade, F., Quiertant, M., Benzarti, K., Dumoulin, J., Aubagnac, Ch. “Nondestructive evaluation of FRP strengthening systems bonded on RC structures using pulsed stimulated infrared thermography”, 193-208.


ABSTRACT

Premature debonding is often observed when strengthening the existing RC structures using externally bonded fibre-reinforced polymer (FRP) sheet. Anchorage system has been proved to efficiently prevent or postpone the debonding from occurring. Although extensive research on the effect of anchorage system on intermediate crack (IC) debonding and end debonding has been carried out, the relevant research on its effect on commonly faced concrete cover separation is still in its initial stage. This paper designed seven beam-type cantilever specimens with different width and height of U-shape end anchorage to investigate its effect on concrete cover (cantilever) separation. The fully separation of cantilever was identified through the knee point of load-strain curves of CFRP laminate at the cracked section. The height of U-anchorage was proved to be more efficient than its width to improve the strength of cantilever separation, while greater width of U-anchorage more efficiently prevented the initiation of intermediate crack induced (IC) debonding of CFRP laminate.

KEYWORDS

CFRP, RC elements, concrete cover separation, cantilever, U-anchorage.

INTRODUCTION

Strengthening by fibre-reinforced polymer (FRP) composites is an effective and convenient method to improve the static and fatigue performance of reinforced concrete (RC) structures. Previous studies (Buyukozturk et al. 2004) have shown that the application of FRP composites to strengthen RC structures can lead to premature debonding failures before the design load is reached. Among the premature debonding failure modes (Smith and Teng 2002), concrete cover separation is often observed. Current models to explain the concrete cover separation of a strengthened beam fall into three broad basis (Zhang et al. 2011, 2012 a, b): (1) the derivation of elastic stress concentrations at the FRP end (2) the shear capacity of strengthened beams and (3) the concrete tooth model.

Existing studies on steel plate or FRP laminate strengthening have proved that the application of anchorage systems is effective to resist the onset and propagation of failure in the external reinforcement-concrete interface or concrete cover to the internal rebar. The most commonly used anchorage types are (Kalfat et al. 2011): 1. Mechanically fastened metallic anchors, 2. U anchors and 3. FRP anchors. Nowadays there have been many studies on the FRP U-anchorage (Khan and Ayub 2011; Jumaat and Alam 2010; Li et al. 2013). Previous research (Smith and Teng 2003) showed that the critical debonding failure mode could be shifted from concrete cover separation to IC debonding with the addition of plate-end U anchorage, further research (Sawada et al. 2003) explained that CFRP U-anchorage resisted the stresses that resulted in cover separation failure.

Most of the previous experimental studies were carried out on the strengthened RC beams with various anchorage systems. The location of cracks which formed the concrete teeth were not pre-determined and thus the effect of geometry of concrete teeth remained unclear and measurement of FRP strain exact at the cracked section through strain gage was difficult. Therefore, a clear and comprehensive understanding of mechanism of U-anchorage and the effect of its designing parameter has not been reached yet.

The authors are conducting a series of studies with the aim at development of an analytical approach for flexural strengthening of an existing structure with external FRP composites with various anchorage systems. The present work concentrates specifically on the use of end U-shape anchorage system and its influence on the mechanism of...
EXPERIMENT PROGRAM

Testing specimens

Seven specimens with different width and height of U-shape end anchorage were prepared. Specimens’ geometry and reinforcement arrangement are shown in Figure 11. The test specimens were 1500 mm long with a rectangular cross section of $150 \times 200$ mm. All specimens had a clear span of 1300 mm, and employed two deformed bars of diameter 10 mm as respective compression and tension reinforcement. 8 mm diameter steel stirrups spaced at 100 mm were placed along the entire length of the beam and the concrete cover thickness at tension and compression side was 46 and 21 mm, respectively, and the side concrete cover was 22. The former value was larger to force the concrete cover separation to initiate. The cantilever sections formed between two adjacent 10 mm wide artificial cracks were 100 mm in length, 150 mm in width and 55 mm in height. The specimens were symmetrical about centre line.

Table 3 Parameters of specimens

<table>
<thead>
<tr>
<th>Group</th>
<th>Label</th>
<th>$b_{uf}$ (mm)</th>
<th>$b_{uh}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L50</td>
<td>L50-60</td>
<td>50</td>
<td>60</td>
</tr>
<tr>
<td></td>
<td>L50-80</td>
<td>50</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>L50-180</td>
<td>50</td>
<td>180</td>
</tr>
<tr>
<td></td>
<td>L100-60</td>
<td>100</td>
<td>60</td>
</tr>
<tr>
<td>L100</td>
<td>L100-80</td>
<td>100</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>L100-180</td>
<td>100</td>
<td>180</td>
</tr>
<tr>
<td>Control</td>
<td>L0-0</td>
<td></td>
<td>Non-anchorage</td>
</tr>
</tbody>
</table>

As shown in Table 1, the specimens were divided into three groups by the width of U anchorage, namely, L50 group, L100 group, and control group. The specimens of L50 group were denoted as L50-60, L50-80 and L50-180, and those of group L100 were labelled L100-60, L100-80 and L100-180. The first number appearing after L represents the width of U anchorage ($b_{uf}$) and the following number indicates the height ($L$) as shown in Figure 11. Specimen L0-0 was a non-anchorage control beam.

Materials

The average compressive strength ($f_{cu}$) of three 150 mm concrete cubes was 26.42 MPa after 28 days of curing. Table 4 shows the mechanical properties of tension and shear reinforcement, where $D$, $E$, $f_y$, $f_u$ denote the unit diameter, the modulus of elasticity, yielding strength and ultimate strength, respectively.

Table 4 Mechanical properties of reinforcement

<table>
<thead>
<tr>
<th>Category</th>
<th>$D$ (mm)</th>
<th>Grade</th>
<th>$E$ (GPa)</th>
<th>$f_y$ (MPa)</th>
<th>$f_u$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10</td>
<td>HRB-400</td>
<td>203.0</td>
<td>493.0</td>
<td>594.5</td>
</tr>
<tr>
<td>2</td>
<td>8</td>
<td>HRB-400</td>
<td>193.0</td>
<td>435.5</td>
<td>639.0</td>
</tr>
</tbody>
</table>

One layer of CFRP consisted of unidirectional textile with a width of 130 mm, length of 900 mm and a dry fibre thickness of 0.111 mm. The elastic modulus and tensile strength of the CFRP laminate was 241 GPa and 3696
MPa, respectively. Adhesive used was a thixotropic epoxy resin with elastic modulus and tensile strength being equal to 2.83 GPa and 52 MPa, respectively.

The following procedure was adopted for specimen preparation. Artificial cracks of 10 mm wide and of height up to the centroid of tension reinforcement was made by four wood plates at casting to form cantilever sections at both sides of specimens. Special attention was paid to make sure one stirrup existing above the centre of cantilever session. All specimens were extracted from the moulds 24 hrs after casting and then cured for 28 days. Three layers of CFRP laminates were then bonded to the soffit of the specimens in a wet-layup procedure after sandblasting, brushing and cleaning of the concrete surface, which was to guarantee an ideal bond between the CFRP and the concrete. Two layers of CFRP U-anchorage were then bonded to the cantilever session in a similar procedure. As indicated in Figure 1, 70mm and 100mm wide CFRP full wrapping was used to prevent fully development of intermediate crack-induced interfacial (IC) debonding before the failure of cantilever. All the specimens were then placed in room environment for seven day for the adhesive curing before testing.

Testing apparatus

The strengthened beam was firstly given a four-point monotonic loading with a shear span of 500mm as indicated in Figure 1. Several strain gages were glued to CFRP laminate (Figure 12) to measure the strain distribution. The deflection was measured by means of LVDTs placed at loading points and supports of the specimens, respectively. Specimens were tested monotonically to failure by a load controlled hydraulic jack at a constant loading rate of 2 kN/min.

Though the parameters of two “teeth” (left and right cantilever) in the same specimen are the same, they may not always fail together due to the scatter. In this case, when the cantilever failed in one side first, the support close to the failed cantilever was moved towards the beam centre and a three-point bending test was conducted as shown in Figure 13 until the failure of cantilever in another side. The locations of LVDTs at two loading points were changed correspondingly.

RESULTS AND DISCUSSIONS

Failure mode

The typical crack distribution of some specimens was recorded as shown in Figure 14.

Two kinds of failure modes, failure of cantilever (concrete cover separation) and flexural failure of beam without cantilever failure were observed for all the specimens. As the load increased, the diagonal shear crack occurred from the top of artificial crack which was nearer to the support, and then horizontal crack appeared and propagated along the level of the steel reinforcement (top line of the cantilever). For control specimen, the cantilever failed in a brittle manner soon after the initiation of horizontal crack, while the load can still increase till the flexure failure of beam. For specimens with U-anchorage height less than 180 mm, the U-anchorage debonded followed by fully separation of cantilever. Instead, for specimens L50-180 and L100-180, neither debonding of U anchorage nor the failure of cantilever was observed, the specimens finally failed in flexure with concrete crushing after the yielding.
of tension reinforcement. As indicated in Figure 4, diagonal crack (No. ③) of concrete cantilever was observed for both L50-180 and 100-180 due to the tensile stress generated by the pull-off force of CFRP laminate.

![Figure 14 Typical cracks distribution (unit: kN)](image)

**Load-strain response (CFRP)**

In order to clarify the failure mechanism of beam-type cantilever with different size of U anchorage, the typical load-strain relationship of CFRP laminate at the cracked section of some specimens is plotted in Figure 15.

As the load increased, the CFRP strain increased linearly followed by some sudden drops which may attribute to the initiation of cracks, cantilever separation or U anchorage debonding. The CFRP strain at the knee point reached the maximum in case of cantilever’s full separation, while the knee point could be surpassed in other possible cases. This knee point was regarded as the judgement of fully separation status of cantilever for all the specimens except specimens L50-180 and L100-180, for which no debonding of U-anchorage and separation of cantilever occurred, and therefore no rapid reduction in CFRP strain was recorded (Figure 5).

![Figure 15 Load-CFRP’s strain correlation curve](image)

**Debonding load**

Both four-point and three-point bending test were conducted for all specimens except L50-80 and L100-180. The corresponding failed cantilever sections and debonding load deduced from load strain curve are listed in Table 5. \( P_{exp} \) represents the average debonding load of four-point and three-point test results. Note that based on the study of Zhang et al. (2011), the load corresponding the concrete cover separation depended upon the distance between the support and the end of CFRP laminate (or location of cantilever in this study), therefore the cantilever failure load (or the moment at the cracked section of cantilever) was considered the same for both A and B side although the shear span was a bit different.
Table 5 Debondig load of specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>A</th>
<th>B</th>
<th>$P_{exp}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L0-0</td>
<td>42.2</td>
<td>42.26</td>
<td>42.23</td>
</tr>
<tr>
<td>L50-60</td>
<td>43.04</td>
<td>48.63</td>
<td>45.84</td>
</tr>
<tr>
<td>L50-80</td>
<td>40.42</td>
<td>43.39</td>
<td>41.91</td>
</tr>
<tr>
<td>L50-180</td>
<td>50.65*</td>
<td>50.24*</td>
<td>50.71*</td>
</tr>
<tr>
<td>L100-60</td>
<td>32.92</td>
<td>45.83</td>
<td>39.38</td>
</tr>
<tr>
<td>L100-80</td>
<td>45.12</td>
<td>41.01</td>
<td>43.06</td>
</tr>
<tr>
<td>L100-180</td>
<td>54.29*</td>
<td>54.7*</td>
<td>54.76*</td>
</tr>
</tbody>
</table>

Note: The value with "*" means ultimate load without cantilever failure.

Figure 16 shows the comparison of debonding load between different specimens. For specimens L50-180 and L100-180, the values represent the ultimate load. For the same height of anchorage, the increase in anchorage width did not increase the debonding load. Actually it was even smaller for anchorage width of 100mm when anchorage height was 60mm. When the anchorage width was 50mm, the debonding load decreased with the increase of anchorage height, while opposite tendency was observed for specimens with anchorage width of 100mm. The debonding load had no clear relationship with the width and height of U-anchorage for those specimens failed in cantilever separation, maybe due to scatter of the test results or insufficient number of test specimens. However, both 50 mm and 100mm wide but 180mm high anchorages can efficiently prevent premature cantilever failure and increase the ultimate load. More tests are expected to be conducted with more options of height of U-anchorage to investigate clearly the effect of U-anchorage after the initiation of cantilever separation.

![Comparison of debonding load](image)

Figure 16 Comparison of debonding load

Mechanism of anchorage

As shown in Figure 7, each concrete tooth acted as a cantilever and the concrete cover separation corresponded to the failure of this cantilever. The load $\sigma_f$ exerted by the longitudinal CFRP is limited by the following factors: (1) the flexural strength of plain concrete: cantilever separates when the concrete tensile stress generates at point A for the section of top end of cantilever reaches the flexural strength. (2) The effect of U-anchorage: the U-anchorage acts as a reinforcement. When the U-anchorage was applied, it prevents the separation from further propagating once it is initiated while the diagonal shear cracks passing through U-anchorage affects the bond strength of U-anchorage and concrete. And the debonding load of anchorage is the maximum “pull out force” at the cantilever section. If this force is larger than the concrete tensile stress generates at point A, the U-anchorage debonds after the initiation of cantilever separation, else the U-anchorage debonds first. With small height of U-
anchorage, the anchorage effect keeps the same since the anchorage debonds right after cantilever separation. Instead, if the height of the anchorage is sufficient, the anchorage strips will not debond and effectively prevent the progress of the cantilever separation. As for different width of U-anchorage with same anchorage height, if the U-anchorage covers the concrete tensile zone, the width of U-anchorage has no obvious difference on the mechanics. Due to the page limitation, the detailed analytical model for prediction of cantilever separation together with Finite Element simulation and experimental verification are not presented here.

CONCLUSIONS

The behaviors of CFRP laminates strengthened beam-type cantilever with or without U-shape CFRP anchorages were experimentally investigated. Several conclusions can be made based on the test results:

1. The load at the fully separation of cantilever can be determined by the key point in the load-strain response curves of CFRP laminate at the end cracked section of the cantilevers.
2. When the height of U anchorage is small, the anchorage debonded before the initiation of concrete cover separation; and the peak load with concrete cover separation kept unchanged. The width of U anchorage had little effect on the concrete cover separation, but greater width of U anchorage can more efficiently prevent the initiation of IC debonding.
3. With sufficient height of U-anchorage, cantilever separation can be prevented, and the peak load was improved.

ACKNOWLEDGMENTS

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REFERENCES


CFRP STRENGTHENING OF RC BEAMS USING A DUCTILE ANCHORAGE SYSTEM

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ABSTRACT

CFRP (Carbon fibre reinforced polymers) systems have been used to strengthen concrete structures during several decades. Especially CFRP plates and rods are used successfully to upgrade existing concrete structures to a required capacity level. Although most of the systems work quite well, several challenges still exist, since the systems show little ductility and the failure modes normally occur through initiation of cracks in the concrete (such as intermediate crack de-bonding). As a consequence the CFRP material is not utilized fully and failure occurs suddenly without sufficient warning. Consequently the de-bonding leaves the concrete beam with a capacity which is of the same magnitude as before strengthening was conducted. This paper presents a strengthening system which improves the utilization of Ø8 and Ø10 CFRP circular rods, increases the ductility by means of a ductile anchorage system and at the same time ensures the required strengthening effect. The initial result shows ductility equal to or larger than the reference T-section beam. The strengthening effect is up to 107%. Extensive research, at the Technical University of Denmark, is ongoing at present time to further verify the results through testing and applied theory.

KEYWORDS

CFRP, RC beams, strengthening, ductility, anchorage.

INTRODUCTION

Strengthening of RC (Reinforced concrete) beams using NSMR (Near surface mounted reinforcement) CFRP (Carbon fibre reinforced polymer) strengthening has been used extensively during the recent years (Mohamed et al. 2008, Rashid et al. 2008, El-Harcha and Rizkalla 2004, Serancino et al. 2007), typically resulting in a significant strengthening effect of the RC structure. However, besides of a susceptibility to fire a major disadvantage related to these strengthening systems is the brittle failure mode and resulting lack of ductility. These failure modes normally initiate as IC (Intermediate crack) de-bonding, (Triantafillou and Antonoplos 2000, Smith 2007, Teng 2003 and Said and Wu 2008), end peeling (Täljsten 1997), concrete cover separation (Gao 2005 et al., Cordon et al. 2008) and sometimes delamination of the fibre material (Cuntze et al. 2004, Schmidt et al., 2012). Investigating these failure modes show that it seems difficult to utilize the full tension capacity of the CFRP material and that the failure is, in addition to this, very dependent on the concrete strength.

Consequently, several efforts to investigate anchoring of the CFRP strengthening system have been going on during the last decade, especially in the field of CFRP plate strengthening, where increased strengthening effects is seen along with an amplified ductility. It is generally accepted that FRP anchoring provides increased efficiency of FRP systems (Smith et al. 2011, Kalfat et al. 2013 and Hansen et al. 2011). However the documentation is still based on limited test results and different test procedures (Kalfat et al. 2013). Anchorage of plates have been used with CFRP dowel anchorages (Smith et al. 2011). This research showed the effect on strength and ductility when placing CFRP dowels in different configurations along the CFRP plate. It showed also that the mid span deflection during loading was significantly affected by the location of the dowels. However it was generally observed that the ductility and strength was increased where the CFRP plate failed in IC de-bonding and shear failure of the CFRP dowel anchors. Also anchoring using steel anchors of multi directional CFRPs is used (Brunckhorst et al. 2007) and patch anchors were seen to increase the ultimate failure load by 39-43% (Kalfat et al. 2013).

The plated systems are relatively easy to anchor in different ways using dowels, concrete steel anchors and nails (Yun et al. 2008), but only limited research have been conducted on anchoring of circular CFRP NSMR systems. The anchoring of such rods often relates to the field of anchorage of tendons for pre- and post-tensioning but these
are also expected to be very efficient for NSMR strengthening. Anchorage of circular CFRP rods remains a challenge, as premature failure modes can occur and cumbersome installation seems to be a common problem (Schmidt et al 2011) compared to conventional methods used for steel rods or cables.

Research at DTU in the field of CFRP anchorage of circular rods during recent years has established an important step towards designing an easy to mount anchorage for CFRP rods (Schmidt et al 2010, 2011). This research has mainly focused on the development of anchorage mechanisms, which utilize the full capacity of the CFRP rods (and plates), but has by now reached the next stage, where the focus is to create an increased ductility of the system, when applied to the RC structures.

The research presented in this paper shows the initial step of this research, where a specially developed ductile end anchorage for 8mm and 10mm circular CFRP rods is applied to T-section RC beams to establish a significantly increased anchoring effect and ductility. The ductile end anchorage system is expected to lead to a more controlled ductile deformation and force transfer, where a contribution from aggregate interlocking from intermediate crack debonding (IC-debonding) can be incorporated.

**EXPERIMENTAL PROGRAM**

The NSMR CFRP circular rod strengthening system was mounted on a T-section RC beam with a total height of 350mm, a flange width of 300mm and a length of 2200mm, Figure 1. 2Ø16 deformed steel bars were used as tension reinforcement and 2Ø8 was placed in the compression zone. Ø8 deformed steel bars was used as shear stirrup reinforcement and placed with a distance of 200mm. The concrete tested according to DS/EN 12390-3 had an average compression strength of 49MPa. The deformed steel reinforcement was tested to have an average yield strength of 569MPa and an E-modulus of 179GPa.

![Figure 1 RC test beam and applied reinforcement.](image1.png)

Figure 2 and Figure 3 shows the location of the 8mm and 10mm circular CFRP rods. The tensile capacity of the CFRP rods was 2500 MPa with an E-modulus of 165GPa according to the manufacturers values. The test configurations are shown in the Table 1.

<table>
<thead>
<tr>
<th>Name</th>
<th>Number</th>
<th>Test configurations and number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference beam</td>
<td>reference</td>
<td></td>
</tr>
<tr>
<td>B-1Ø10U*</td>
<td>1, 2</td>
<td>Two beams with one circular Ø10 NSMR CFRP</td>
</tr>
<tr>
<td>B-2Ø10U</td>
<td>1, 2, 3</td>
<td>Three beams with two circular Ø10 NSMR CFRP</td>
</tr>
<tr>
<td>B-2Ø8M</td>
<td>1, 2</td>
<td>Two beams with two circular Ø8 anchored NSMR CFRP</td>
</tr>
<tr>
<td>B-2Ø10M</td>
<td>1, 2, 3</td>
<td>Three beams with two circular Ø10 anchored NSMR CFRP</td>
</tr>
</tbody>
</table>

* (Explanation, B-1Ø10U – B: Beam: 1:one CFRP rod, Ø10:10mm in diameter, U without anchor (M: with anchor))

![Figure 2 Location of one circular CFRP NSMR rod in the T-section bottom concrete cover.](image2.png)

![Figure 3 Location of two circular CFRP NSMR rod in the T-section bottom concrete cover.](image3.png)
ANCHOR MOUNTING

Anchoring of the CFRP NSMR rod is shown in Figure 4, where a specially designed anchor configuration is mounted to the RC beam ends. The mounting procedure was as follows: 1) mounting of thread M24 bars into the RC beam, 2) pre-mounting of mechanical anchor on the CFRP rod (The CFRP rod was already glued into the RC beam leaving the ends exposed for the anchor installation), 3) mounting of anchor block, 4) applying fast curing glue (to ensure sufficient CFRP/wedge contact between CFRP and inner wedge surface) and 5) pre-setting of wedge into the barrel. The mounting procedure of the anchored strengthening system lasted approximately 10 min.

Figure 4 Installation of the specially designed anchor system.

TEST SETUP AND MONITORING

The beams were loaded using deformation controlled actuators (2mm/min) in a four point bending setup, Figure 5, where strain gauges (SG) were applied on the CFRP rods, deformed steel reinforcement and concrete. Also LVDT transducers (DF) were added to investigate the beam deformations during loading. All RC beams were designed to fail in bending, in order to ensure sufficient activation of the ductile anchorage system and thus enable optimal verification of the system response.

Figure 5 Test setup and applied monitoring equipment (view from side and below).

RESULTS AND DISCUSSION

Figure 6a shows the failure mode of the un-anchored NSMR CFRP rods, where IC-debonding failure was experienced for all beams. Such a failure mode is typically obtained when using these strengthening systems and occurs suddenly without much warning. As seen from Figure 6b, where the anchored beams are shown, a different
failure mode was obtained, which is characterised by significant cracking and concrete peeling, and an increased deformation before concrete crushing occurs.

Using one and two 10mm rods without anchors, resulted in a maximal deflection (at the loading points) of 19mm and 12mm respectively. However, when anchoring the 10mm rods, the maximal deflection was increased to 46mm (287% increase). Figure 7 shows representative beam deflection shapes at different load steps for an un-anchored system (Figure 7a) and an anchored system (Figure 7b), both with 10mm CFRP rods. It is seen how the anchored configurations have a symmetrical deflection, which is significantly larger than the obtained in the un-anchored configurations. This correspond to the authors expectations of developing a sustained- and controlled failure mechanism with increased ductile behaviour, just as the aggregate interlocking seems to contribute to the beam improved performance.

When evaluating the strain development in the CFRP rods, the un-anchored CFRP rods showed an un-symmetrical development of strains during loading when delamination initiate, Figure 8a. However, when the CFRP rods were anchored the strain development continued to be symmetrical throughout the loading regime, Figure 8b, indicating the desired activation of the ductile anchors.

The load/moment - deformation curve for each configuration is depicted on Figure 9. A significant strengthening effect is generally obtained by using the NSMR CFRP rods. The un-anchored configurations, however, show an almost linear response until a sudden failure occurs and consequently a low ductility. When the ductile anchors are applied to the NSMR CFRP rods, a significant increase of the capacity was obtained, but with substantial ductility as the outcome as well. The linear response ends at a moment of approximately
100kNm (approximately 70kNm for the reference beam) where a horizontal regime, with a small inclination starts to develop. This regime stays almost stable until concrete failure occurs. The load/moment versus deflection are shown in Figure 9a. Figure 9b shows an example of the ductile beam response when anchoring two 8mm NSMR CFRP rods. At a deflection of approximately 20mm and 35mm sudden reductions of loads occurred. Generally, these reductions are dedicated to the intended movements in the ductile anchor setup and concrete cracking (aggregate interlocking failure), where sliding of the anchorage parts occur during loading in the horizontal deformation regime.

**Figure 9**

a) Representative load/moment – deformation curves for the test configurations. b) Example of sudden sliding in the ductile anchor system.

The ductile anchor setup behaviour in combination with aggregate interlocking seems to provide the full ductile beam response. Consequently, this mechanism offers substantial warning and ductility. An overview of the results is seen in Table 2. The theoretical moment capacity of the T-section RC reference beam is calculated to be 64.3 kNm and ACI (ACI 2008) is used to calculate the strengthening magnitude.

<table>
<thead>
<tr>
<th>Test configuration</th>
<th>Test [kNm]</th>
<th>ACI [kNm]</th>
<th>Deviation [%]</th>
<th>Capacity increase [%]</th>
<th>Deflection [mm]</th>
<th>Deflection increase [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-2Ø10U-1</td>
<td>120,7</td>
<td>134,1</td>
<td>10</td>
<td>88</td>
<td>15</td>
<td>-58%</td>
</tr>
<tr>
<td>B-2Ø10U-2</td>
<td>133,0</td>
<td>134,1</td>
<td>1</td>
<td>107</td>
<td>13</td>
<td>-64%</td>
</tr>
<tr>
<td>B-2Ø10U-3</td>
<td>119,5</td>
<td>134,1</td>
<td>11</td>
<td>86</td>
<td>16</td>
<td>-56%</td>
</tr>
<tr>
<td>B-2Ø8M-1</td>
<td>111,8</td>
<td>129,2</td>
<td>13</td>
<td>74</td>
<td>52</td>
<td>44%</td>
</tr>
<tr>
<td>B2Ø8M-2</td>
<td>121,7</td>
<td>129,2</td>
<td>6</td>
<td>89</td>
<td>52</td>
<td>44%</td>
</tr>
<tr>
<td>B-2Ø10M-2</td>
<td>130,6</td>
<td>152,7</td>
<td>14</td>
<td>103</td>
<td>47</td>
<td>31%</td>
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<tr>
<td>B-2Ø10M-2v2**</td>
<td>149,1</td>
<td>152,7</td>
<td>2</td>
<td>132</td>
<td>31</td>
<td>-14%</td>
</tr>
<tr>
<td>B-2Ø10M-3</td>
<td>121,7</td>
<td>152,7</td>
<td>20</td>
<td>89</td>
<td>43</td>
<td>19%</td>
</tr>
</tbody>
</table>

* The deflection of the reference beam was measured to 36mm
**B-2Ø10M-2 failed initially at the support. The beam was then tested further as v2 (second version) to examine the final bending capacity

**CONCLUSIONS**

Four test series were investigated using ductile anchored and un-anchored 8mm and 10mm CFRP NSMR circular rods. The research is the first step in a large research program aiming at developing a strengthening system with a controlled ductile deformation, increased warning, and a higher strengthening effect. The following can be concluded from the test programme.

- The developed ductile anchor setup was relatively easy to apply to the RC T-section beams. This opens a realistic opportunity of mounting the system on real life structures.
- Using the anchored NSMR lead to a different failure mode, where significantly increased ductility and warning is obtained, compared to the un-anchored NSMR configurations.
- The CFRP material was activated equally well in both ends when the anchor setup was applied. This enabled a better utilization of the rod.
- It seems possible to secure a controlled ductile behaviour by using the developed anchor setup
- Generally, the anchor setup worked as required in the test programme.
The result shows an increased capacity of 132% and a maximal deformation increase of up to 287% when using anchored 10mm NSMR CFRP circular rods compared to un-anchored rods. Additional research is required in order to investigate the failure mechanisms, optimize the anchor setup, apply the system to longer beams, etc. and is presently ongoing at DTU.

ACKNOWLEDGMENTS

The authors gratefully acknowledge Condor Kemi and Sto Scandinavia who have supported the project with materials and information.

REFERENCES


NUMERICAL ANALYSIS OF BENDING AND SHEAR IN LARGE-SCALE PRE-TENSIONED CONCRETE BRIDGE GIRDERS STRENGTHENED WITH PRE-STRESSED CFRP LAMINATES

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ABSTRACT

This paper presents the research activities performed within a Polish-Swiss Research Program between EMPA (Switzerland) and Lodz University of Technology (TUL, Poland) on two large-scale post-tensioned concrete girders. The main aim of the project was a pioneer application of an innovative strengthening system with pre-tensioned carbon fibre reinforced polymer (CFRP) strips on an existing bridge over the Pilsia River in Szczercowska Wieś (central Poland). The project consisted of the laboratory tests of two post-tensioned concrete beams, which were performed as the exact copies of the original bridge girders. The first of them served as a reference member, while the second one was strengthened for flexure and for shear with pre-tensioned externally bonded CFRP reinforcement. The paper presents the main test results and the numerical model created in a commercial FEM software Atena, considered one half of the symmetrical girder, with a step by step analysis of several loading phases of the tested girder referring to the self-weight of the girder, post-tensioning, slab casting, strengthening, and external loading until failure. The comparison of the test and numerical load-deflection response shows a very good agreement and promising results for both flexure and shear.

KEYWORDS

FRP, pre-stressed, concrete, bridge girder, FEM, analysis, flexure, shear, capacity.

INTRODUCTION

Flexural strengthening of RC structures with CFRP laminates is actually a well-recognised technique of improving load capacity, although this method does not fulfil the serviceability limit state (SLS) of the strengthened structures. In order to improve this condition, pre-tensioning of CFRP reinforcement is recommended. In case of pre-stressed or post-tensioned (PT) concrete structures the only effective strengthening technique is the application of the pre-tensioned CFRP laminates. Due to the fact that pre-stressed tendons are located near the bottom surface, such structures cannot be strengthened with the use of steel plates anchored with mechanical anchors because they could cut the pre-stressing steel reinforcement during the mounting of the anchors on the post-tensioned beam. To solve this problem a unique gradient strengthening method has been proposed (Meier et al. 2001, Czaderski et al. 2012, Michels et al. 2013). Prior to the first application of this technique on the bridge over the Pilsia River in Poland (Kotynia et al. 2015), an extensive experimental research program had been conducted within the Polish-Swiss Tulcoempa project (Michels et al. 2016). The main aim of the paper is to present a new FE-model using ATENA software and to apply it for an analysis of the behaviour of the PT girders tested in this project, both in terms of flexure and shear. The results of the experimental tests of two PT girders, strengthened with pre-tensioned laminates were used to verify the proposed MES model.

EXPERIMENTAL PROGRAM

Two full-scale I-shaped girders 18.4m in length and 1.0m in height were reconstructed according to the original bridge drawings and then cast of class C35/45 concrete (of 61.4MPa and 62.1 MPa compressive concrete cube strength of girders 1 and 2, respectively). Then girders were post-tensioned with three parabolic and two straight steel pre-stressing tendons with a total cross section of 345 mm² (each tendon pre-stressed with the pre-stressing force of 363 kN, the average yield and ultimate strength of 1.660 MPa and 1.810 MPa and the ultimate strain of 4.8%).
The last stage of prefabrication of the girders was casting the new upper concrete deck (125 cm wide and 21 cm deep, with the compressive concrete cube strength of 47.5 MPa and 51.1 MPa in girder 1 and 2, respectively) that finally gave the T-section PT concrete girder shown in Figure 1.

The first girder served as a reference, while the second one was strengthened in flexure with two pre-tensioned CFRP laminates (100 mm wide, 1.2 mm thick, with the modulus of elasticity equal to 165 GPa), with a gradient anchorage method (without mechanical anchors applied with three consecutive force releases equal to 50 kN (over the first of 300 mm bond length) and two next releases of 35 kN (each over 200 mm bond lengths)). After the flexural strengthening, the shear strengthening was applied. The shear region was strengthened with 14 vertical wrappings of CFRP sheet (of 75 mm width and about 0.9 mm mm thick, with spacing of 150 mm, with the modulus of elasticity equal to 240 GPa) applied with the wet lay-up technique (Figure 2). A detailed description of the reconstruction of the girders and the strengthening process is presented in (Michels et al. 2016).

Experimental tests

In order to investigate the strengthening efficiency, the girders were tested in a 6-point bending set-up. The supports were set at 18.0 m span and four actuators were situated symmetrically in the midspan of the beam at 1.2 m spacing. The loading configuration was selected so as to be compatible with the adequate Polish Standard (PN-85/S-10030 code for bridge design) for the calculation of the designed loads for concrete bridges. The loading was controlled by displacement of the hydraulic jacks and set to 3.5 mm/min. In the first phase the girders were preloaded with 4 x 100 kN force, then unloaded and loaded again until failure (Figure 3).

The reference girder reached its ultimate load of 4 x 193 kN and failed due to extensive yielding of the prestressing tendons. The actual failure (concrete crushing or rupture of the reinforcement) was not reached, but the tests had to be stopped due to very high deflections. However, the load-deflection response obtained from the test (Figure 4) allowed to assume that the ultimate load was not likely to increase significantly, should the test be continued. The strengthened girder failed due to the rupture of both longitudinal CFRP laminates. The ultimate load was equal to 4 x 240 kN and the strain in the CFRP reinforcement reached 1.6% at rupture (including pre-strain). The complete results of the experimental tests are presented in Table 1.
Table 6 Experimental tests results.

<table>
<thead>
<tr>
<th></th>
<th>Experimental tests</th>
<th>Numerical simulations</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Girder 1 flexure</td>
<td>Girder 2 flexure</td>
</tr>
<tr>
<td>Ultimate load, $F_u$ [kN]</td>
<td>772</td>
<td>960</td>
</tr>
<tr>
<td>Ultimate bending moment, $M_u$ [kNm]</td>
<td>3461*</td>
<td>4194*</td>
</tr>
<tr>
<td>Cracking load, $F_{cr}$ [kN]</td>
<td>380</td>
<td>440</td>
</tr>
<tr>
<td>Max. deflection, $u_{max}$ [mm]</td>
<td>260</td>
<td>210</td>
</tr>
<tr>
<td>Max. concrete compressive strain, $\varepsilon_{cc}$ [%]</td>
<td>0.23</td>
<td>0.15</td>
</tr>
<tr>
<td>Max. concrete tensile strain, $\varepsilon_{ct}$ [%]</td>
<td>0.87</td>
<td>0.94</td>
</tr>
<tr>
<td>CFRP pres-strain, $\varepsilon_{fp}$ [%]</td>
<td></td>
<td>0.60</td>
</tr>
<tr>
<td>CFRP strain increase during test, $\varepsilon_{f,test}$ [%]</td>
<td>1.00</td>
<td></td>
</tr>
</tbody>
</table>

*including self weight  
**not including permanent deformation after flexural test

Shear tests

No signs of shear failure have been observed during flexural tests, even though the loads and deflections were extremely high. For a direct verification of the girders’ shear behaviour, an additional experimental test was conducted, in which the strengthened girder was stripped of all the CFRP reinforcement and subjected to high shear loads. The goal of the tests was to confirm that, contrary to traditional strut-and-tie approach (chapter numerical model), a direct transfer of the compressive concrete stresses due to the arch effect from the loading point to the support prevented the shear failure and provided the girders to carry much higher loads, even without any shear strengthening.

For the purpose of the extended tests of the strengthened girder (previously tested for flexure), all the CFRP reinforcement had been removed and the loading points were moved as close to one of the supports as it was possible due to the loading setup. The spacing between loading points was the same as in the flexural tests (4 x 1.2m) and the shear span of 2.2 m (Figure 4).

![Figure 4 Shear test scheme](image)

Even in the second test the girder did not fail in shear. The maximum load applied reached 4 x 268 kN, which corresponded to the highest shear force in the shear span equal to 938kN. The test was stopped due to high deflections and yielding of the longitudinal reinforcement. The deflection in the girder’s midspan reached 185 mm (not including permanent deformation resulting from the flexural test). The highest diagonal compressive strain in concrete registered with digital image correlation system Aramis reached 0.6%. The results of the shear test are shown in Table 1.

NUMERICAL ANALYSIS

An exact model of the laboratory girders was created in the FEM software Atena 2D. The constitutive laws of the model comprised: a non-linear behaviour of concrete in compression, non-linear fracture mechanics, Kupfler biaxial failure criterion, the reduction of concrete compressive strength, shear stiffness after cracking and fixed crack direction (compatible with principal stresses direction). Real mechanical properties of all constitutive materials (concrete, pre-stressing and mild steel, CFRP reinforcement) were implemented in the model, based on laboratory tests or manufacturer’s data. One half of the girder was modelled for the simulation of flexural tests, with appropriate symmetry boundary conditions in the middle. For the simulation of the shear test, the whole girder was modelled due to lack of symmetry in the loading pattern (Figure 5).
To reproduce the internal stresses in the girder correctly, multiple loading cases and steps were incorporated in the model. Initially, during the post-tensioning, the girder lied on its formwork. That boundary condition was also given in the simulation in order to avoid bending moment and premature bending cracks caused by the self-weight of the girder. However, for the external loading that line support had to disappear. Since the line supports incorporated in the program cannot be removed during the simulation, a non-linear contact spring was modeled as line support instead. That contact spring was ineffective for tensile forces and compressive forces higher than those due to self-weight. Therefore the girder could deflect freely upwards during post-tensioning (later during external loading the contact spring was switched off). To simulate the post-tensioning, an adequate tensile force was introduced in each tendon. At that time, the weight and modulus of elasticity of concrete in the upper slab in the model was reduced to 0, as the slab of the laboratory girder cast after the post-tensioning. In the second phase the actual weight and stiffness of the concrete slab was applied, which corresponded to its casting. In further steps the girder was externally loaded by prescribed displacement of a mass-less load distribution beam with two contact points with the girder (representing two loading jacks). The displacement was increased by 5 mm (and later 10 mm) in each step.

The comparison of experimental and numerical load-deflection and load-strain curves shows an excellent compatibility of laboratory tests and FEM simulations (Figure 6). The numerical models of both girders showed behaviour identical with the behaviour of the tested members, up to the point where the failure occurred or the tests were stopped. In the simulation of the reference girder’s flexural test, at 260 mm midspan deflection the applied external load was an equivalent of $4 \times 190$ kN, which was almost exactly the same as in the laboratory test.
The simulation of further loading made it possible to obtain a failure due to concrete crushing under the load of 4 \times 196 \text{kN} and midspan deflection equal to 437 \text{mm}. In case of the strengthened girder’s flexural simulation, the load corresponding to CFRP laminates’ rupture (\(\varepsilon_{\text{f,test}} = 1.0\%\)) was equal to 231\text{kN} (at 203 \text{mm deflection}), which corresponds very well with the ultimate load of 4 \times 240 \text{kN} (at 210 \text{mm deflection}) obtained in the laboratory test. Similar consistency was also achieved in terms of internal strains and stresses in the girders, as well as cracking pattern and inclinations of cracks (see Table 1, Figure 6).

The simulation of the shear test confirmed very high shear load capacity of the girder. The applied loads reached similar levels as in laboratory test. Maximum external load in the model was equal to 4 \times 270 \text{kN}, which corresponded to support reaction equal to 938 \text{kN}. The directions and values of principal stresses (Figure 7) show the phenomena of direct transfer of compressive loads from loading points to the supports and pre-stressing cables anchors via concrete compressed diagonally. The average compressive stress in concrete diagonal was equal to about 21 \text{MPa} and the principal compressive strain reached 0.057\%. The simulation also showed that even though almost all the stirrups in the shear span were yielded, the shear stresses were effectively transferred via compressed

Figure 7 a) Stress pattern in girder 1, b) cracking pattern in flexural test of girder 1, c) cracking pattern in the flexural test of girder 2, d) cracking pattern in shear test of girder 2.
concrete diagonal and pre-stressed cables. Similarly to the results of flexural simulations, almost identical cracking patterns were obtained in the real shear test and simulation (Figures 7b and 7c).

**CONCLUSIONS**

On the basis of a comparative analysis of the test and numerical results, the following conclusions can be drawn:
- Very good compatibility of flexural test and numerical results
- Excellent consistency of the MES model and test results in terms of the ultimate loads, midspan deflection, concrete strain, and the cracking pattern.
- The proposed FE model successfully predicts enhancing effects in the ULS and SLS in the full range of loading.
- The proposed model can be used for parametric simulations considering the following investigated parameters: variable pre-stressing force in the internal pre-stressing tendons; variable pre-stressing level of the externally bonded pre-stressed laminates; variable elasticity modulus of FRP laminates and concrete strength; flexural behavior of PC girder strengthened with CFRP laminates without pre-stressing; flexural behavior considering the shrinkage in the slab (top deck); shear behavior without concrete filling elements.

**REFERENCES**


CONTRIBUTION OF EXTERNALLY BONDED FRP TO FLEXURAL CAPACITY OF REINFORCED MASONRY WALLS SUBJECTED TO OUT-OF-PLANE LOAD

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ABSTRACT

Masonry walls have failed out of plane much more frequently than they have in plane; however, more research and investigations have aimed to improving in plane capacity. The objective of this study is to investigate the out-of-plane flexural performance of reinforced masonry walls and evaluate the contribution of externally bonded fiber-reinforced polymer (FRP) sheets and strips to flexural capacity. Nine fully grouted reinforced masonry walls were built using concrete masonry units and type S mortar. The out-of-plane loading was simulated by four-point bending with an effective span of 1.12 m (44 in.) between the supports. The reinforced walls subjected to cyclic load at a rate 1.27 mm/min (0.05in./min). The reference masonry wall was in running bond pattern and reinforced with 2 No. 4 steel bars. The following experimental parameters were investigated: the type and amount of FRP composite, steel reinforcement ratio, and the effect of surface preparation. The test setup, instrumentation of the specimens and the discussion for the results and modes of failure are described in this study. The general behavior of the reference and strengthened specimens are characterized by load–deflection curve responses. Different modes of failure, including compressive concrete crushing failure, FRP rupture, and FRP reinforcement debonding from the masonry substrate occurred in the strengthened reinforced walls.

KEYWORDS

Reinforced masonry walls, Strengthening, FRP, EB, out-of-plane, cyclic load.

INTRODUCTION

There is a lot of existing masonry structures that do not satisfy design and structure lifetime requirements due to aging, change in use and corrosion. For these reasons, masonry walls that have an insufficient out-of-plane strength to resist the forces generated by seismic events, wind, or lateral earth pressure are in need of an upgrading capacity to extend their lifetime. Fiber reinforced polymer (FRP) has become an attractive method for strengthening masonry structures. (Al-Jaberi et al., 2015) evaluates the behavior of reinforced masonry wall strengthened with NSM-FRP bars. The results of this study showed that the NSM technique affects the flexure strength and mode of failure for the tested specimens. This technique is affected by FRP properties, FRP surface treatment, groove size and geometry, bar size and adhesive agent (Sharaky et al., 2014). Significant research has focused on improving out of plane behavior of unreinforced masonry wall system by using external bonded FRP. Unreinforced masonry walls strengthened using GFRP strip and subjected to out of plane cyclic load was investigated by (Velazquez-Dimas et al., 2000). This study showed that the Strength and deformation capacity of the retrofitted wall were significantly enhanced, retrofitted wall resisted pressure was approximately five times the wall weight and the deflection as much as 5% of the wall height. The results of experimental research on brick masonry vaults strengthened by fiber-reinforced polymer (FRP) strips are presented (Valluzzi et al., 2001). The presence of the fibers prevents the typical brittle collapse of brick masonry. Three failure mechanisms are possible depending on the position and amount of the reinforcement in the strengthened masonry structure: (1) masonry crushing, (2) detachment of the fibers; and (3) sliding along a mortar joint due to the shear stresses. The cyclic behavior of unreinforced brick walls, before and after retrofitting using Glass Fiber Reinforced Polymers (GFRPs) was investigated (Kalali and Kabir, 2012). These experimental tests demonstrate the ability of GFRPs to keep the bricks together and maintain the specimens integrity in addition to prove that GFRP provide an effective technique for improving the seismic resistance of unreinforced brick walls.
The vast majority of previous studies have focused on the behavior of unreinforced masonry walls. The research reported here extended the previous studies by considering the behavior of fully grouted reinforced masonry walls strengthened with different types of FRP under half reversed cyclic loading. Nine reinforced masonry walls were strengthened externally using GFRP sheets and CFRP laminate. The parameters considered were the type and amount of FRP composite, steel reinforcement ratio, and the effect of surface preparation.

**EXPERIMENTAL PROGRAM**

**External Strengthening System**

The procedures for installing FRP systems may differ slightly depending on manufacturers that developed the system. Temperature and surface moisture in addition to surface preparation of substrate at the time of installation are the effective parameters on performance of FRP systems.

The GFRP sheet to be glued on the surface is 200 mm (8 in) width and 0.36 mm (0.014in) thick in size. Prior to attaching FRP sheet to the tension surface of reinforced masonry walls, a primer coat of epoxy was applied to the specimen in the regions where the GFRP sheets were to be bonded. The GFRP sheets was saturated in epoxy and then positioned by hand on the prepared surface and a paint roller was used to release trapped air below the fiber sheet. This process of strengthening named wet lay-up method can be seen in “Figure 1” below.

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**Test Specimens**

Nine reinforced masonry specimens were constructed using standard masonry blocks 152.5 mm (6 in.) and type S mortar. The nominal dimensions of these walls were 1220 mm (48 in.) six course length by 610 mm (24 in.) one and a half course width as shown in “Figure 2”. The specimens were constructed in a running bond and reinforced with different of vertical steel reinforcement ratio. The walls were. They were grouted four days after construction to ensure stability during the vibration process. After a curing period, the walls were strengthened using a GFRP composite (unidirectional E- glass fabric with an epoxy matrix) and a CFRP laminate. The primary variables for this study are listed in Table 1.

---

**Specimen Designation**

(a) Masonry walls before strengthening  (b) Eternally bonded specimen

Figure 2 Masonry Specimens
The specimen ID consisted of three parts as shown in Table 1. The first part represented FRP information (type, thickness, and width). The first character identified the FRP type: namely “C400” for carbon FRP laminate and “G” for SHE-51 composite (E-glass). The second and third characters referenced the cross-section thickness and width in mm, respectively. The second part of the ID identified the bonding adhesive type, number of FRP strips in the tension face, and the wall bond pattern. The first character represented the bonding adhesive type: E (for Tyfo S Epoxy) and S (for SikaDur 30). The second character identified the number of strips in the tension face. The third character referred to the wall bond pattern applied: “R” for running bond. For specimen No.2, the letter W refers to the specimen without surface preparation. The third part of ID represents the main steel reinforcement.

<table>
<thead>
<tr>
<th>Wall</th>
<th>Specimen ID</th>
<th>Type of FRP</th>
<th>Bonding Adhesive</th>
<th>Number of Strips</th>
<th>surface preparation</th>
<th>steel reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Control</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2#4</td>
</tr>
<tr>
<td>2</td>
<td>Control</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2#3</td>
</tr>
<tr>
<td>3</td>
<td>G(1.3)200-E1RW</td>
<td>GFRP</td>
<td>Tyfo S epoxy</td>
<td>1</td>
<td>Without Filler</td>
<td>2#4</td>
</tr>
<tr>
<td>4</td>
<td>G(1.3)200-E1R</td>
<td>GFRP</td>
<td>Tyfo S epoxy</td>
<td>1</td>
<td>Epoxy Putty Filler</td>
<td>2#4</td>
</tr>
<tr>
<td>5</td>
<td>G(1.3)200-E1R-2#3</td>
<td>GFRP</td>
<td>Tyfo S epoxy</td>
<td>2</td>
<td>Epoxy Putty Filler</td>
<td>2#3</td>
</tr>
<tr>
<td>6</td>
<td>G(1.3)200-E1R-1#5</td>
<td>GFRP</td>
<td>Tyfo S epoxy</td>
<td>2</td>
<td>Epoxy Putty Filler</td>
<td>1#5</td>
</tr>
<tr>
<td>7</td>
<td>G(1.3)200-E2R</td>
<td>GFRP</td>
<td>Tyfo S epoxy</td>
<td>2</td>
<td>Epoxy Putty Filler</td>
<td>2#4</td>
</tr>
<tr>
<td>8</td>
<td>C400(1.4)50-S1R</td>
<td>CFRP</td>
<td>SikaDur30</td>
<td>1</td>
<td>-</td>
<td>2#4</td>
</tr>
<tr>
<td>9</td>
<td>C400(1.4)50-S2R</td>
<td>CFRP</td>
<td>SikaDur30</td>
<td>2</td>
<td>-</td>
<td>2#4</td>
</tr>
</tbody>
</table>

Specimen Surface Preparation and FRP Installation

The surface was cleaned manually with a wire brush to remove all excessive mortar from the walls joints that left from the construction process. The prepared surfaces were vacuumed after brushing to remove the residual dust. Tyfo S epoxy resin mixed with silica fume to provide a putty filler layer that smoothed and leveled the prepared surface before composite material was installed. The SEH51 fabric was saturated with Tyfo S epoxy resin before it was applied to the wall. The saturation process ensured good bonding with the substrate. The Tyfo S epoxy resin was mixed at a volume ratio of 100 parts A to 42 parts B. The curing period for Tyfo S epoxy resin is three days at 60o C. SikaDur 30 adhesive used to bond the Aslan 400 CFRP strip. SikaDur30 mixed with a volume proportion of one part of component B to three parts of component A. The FRP sheet or laminate bonded to the tension face of the wall so that the fiber was perpendicular to the bed joints.

Material Properties

A series of tests was performed to determine each material’s mechanical properties. The properties of the materials that were used to construct the specimens are summarized in Table 2. The manufacturing properties of FRP and its bonding adhesive are presented in Table 3.

<table>
<thead>
<tr>
<th>Item</th>
<th>Properties</th>
<th>Values (MPa)</th>
<th>Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete block</td>
<td>Prism compressive strength</td>
<td>21</td>
<td>ASTM C1314-12</td>
</tr>
<tr>
<td>Mortar type S</td>
<td>Compressive strength</td>
<td>17.5</td>
<td>ASTM C109-13</td>
</tr>
<tr>
<td>Grout</td>
<td>Compressive strength</td>
<td>35</td>
<td>ASTM C1019-13</td>
</tr>
<tr>
<td>Steel bar</td>
<td>Yield strength</td>
<td>471</td>
<td>ASTM A370-13</td>
</tr>
<tr>
<td></td>
<td>Modulus of Elasticity</td>
<td>203000</td>
<td></td>
</tr>
</tbody>
</table>

Note: 1.0 MPa = 145 psi.
Table 3- Mechanical properties of FRP and bonding adhesive

<table>
<thead>
<tr>
<th>Type of FRP</th>
<th>Thickness (mm)</th>
<th>Ultimate tensile strength (MPa)</th>
<th>Elongation at break %</th>
<th>Tensile Modulus (MPa)</th>
<th>Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>SHE-51 composite (E-glass)</td>
<td>1.3</td>
<td>575</td>
<td>2.2</td>
<td>26100</td>
<td>ASTM D3039-14</td>
</tr>
<tr>
<td>Aslan 400 CFRP Strip</td>
<td>1.4</td>
<td>2400</td>
<td>1.87</td>
<td>131000</td>
<td>ASTM D3039-14</td>
</tr>
<tr>
<td>SikaDur 30</td>
<td>-</td>
<td>24.8</td>
<td>1</td>
<td>4482</td>
<td>ASTM D638-14</td>
</tr>
<tr>
<td>Tyfo S epoxy</td>
<td>-</td>
<td>72.4</td>
<td>5</td>
<td>3180</td>
<td>ASTM D638-14</td>
</tr>
</tbody>
</table>

Note: 1.0 GPa = 145.03 ksi; 1.0 MPa = 0.145 ksi; 1.0 mm/mm = 1.0 in./in.; 1.0 mm = 0.039 in.

Test Setup and Loading System

Four-point line loading with simply supported boundaries can be used to conduct out-of-plane testing of reinforced masonry walls as shown in “Figure 3”. An MTS double-acting hydraulic jack with a push-pull capacity of 965 MPa (140 kips) was used to apply the load. The specimens subjected to out of plane loading without any consideration to normal (vertical) load that available in case of bearing walls, so the behavior doesn’t include such effect. This load was transferred to the masonry specimen by means of continuous steel plates and bars along the full width of the external face of reinforced walls to provide two equal line loads. The distance between these two lines was 200 mm. (8 in.). The load was applied in half-cycles of loading and unloading, as a displacement control at a rate of 1.27 mm/min (0.05 in./min). The displacement amplitude increment was 6.35 mm (0.25 in.), double half loading cycle was applied for each amplitude level as illustrated in “Figure 4”. Deflections at mid and third span were measured using three (LVDTs) at each side. FRP and steel bars strain are recorded during the loading using strain gauges placed on the FRP and steel bars.

EXPERIMENTAL OBSERVATIONS

Reinforced concrete masonry walls generally behave in a flexural ductile mode due to steel reinforcement. The first flexural tensile crack initiated at the block mortar in the maximum moment region (between two line loads). Further flexural tensile cracks in the block mortar and within the CMU developed beyond the cracking load. The FRP sheets that attached to the masonry caused cracks to propagate in the masonry units. The masonry cracks were oriented parallel to the bed joints. Cracks also extended along the FRP sides due to high stress in this region. The cracks also moved vertically toward the compression face in a straight line, as illustrated in “Figure 5”. Flexural shear cracks outside the constant moment region generated in the later stages of loading. A number of distinct failure modes of reinforced masonry walls strengthened externally by FRP observed in this experimental study. The first mode was flexural failure (FRP rupture or concrete crushing) and the second mode was the FRP debonding from the masonry wall. The composite action between the masonry wall and FRP ended at the failure. The debonding failure in all its forms, whether intermediate crack (IC) or plate end-debonding failure generally happened in the masonry substrate. A strong adhesive between the GFRP and masonry would prevent
the debonding failure between the adhesive and concrete masonry or between the adhesive and FRP. Pictures of typical modes of failure are shown in “Figure 6”.

![Image of typical modes of failure]

**Figure 5** Cracks developed during loading

**Figure 6** Observed mode of failure

---

### EXPERIMENTAL RESULTS AND COMPARISON

The load vs. deflection curves envelopes are illustrated in “Figure 7”. The moment capacity and stiffness of the reinforced walls were increased for specimens externally strengthened with FRP (compared to the control specimen). Interestingly, the flexural capacity of walls strengthened with one strip of FRP composite dropped to approximately the same capacity of the control specimen when the failure (debonding) occurred. On the other hand, the flexural capacity of walls strengthened with two strips of FRP dropped below the control specimen capacity. This is due to existing of block unit cracking and damage that accumulated after failure occurred.

It is worthwhile to point out that in a general sense, using putty filler layer lead to increase the shear capacity for the walls, which can be attributed to the improvement of masonry unit as a result of absorption of epoxy.

The load-deflection responses for the specimens strengthened with GFRP composite and CFRP laminate are illustrated in “Figure 7 a and b”, respectively. Strengthening the specimens with GFRP composite is increased load carrying capacity by 300%, while the CFRP increased the specimen’s capacity by 81%. The reason for this difference is the bonding material. Using silica fume mixed with Tyfo S epoxy resin in specimens with GFRP reduced the porosity of the concrete unit and increased its compressive strength. The specimens with GFRP and epoxy resin showed a better behavior than the specimens with CFRP and SikaDur30 due to the epoxy’s high debonding strain compared with SikaDur30, as shown in “Figure 7 c and d”. It’s very limited experimental studies have considered the effect of varying longitudinal steel ratio on the behavior of upgraded masonry walls. “Figure 7 e” shows this effect, the control capacity and the initial stiffness affected by the longitudinal reinforcement. The stiffness depend on external strengthening and the internal reinforcement ratio, so the stiffness of specimen reinforced with 2#4 more than other reinforced with 2#3 bars. The ultimate load and post peak behavior depend on controlling mode of failure which is independent of the steel reinforcement ratio. The specimen reinforced with 2#4 failed by rupture of FRP followed by masonry crushing that lead to post peak ductile behavior. On the other hand, the specimen reinforced with 2#3 failed by plate end debonding followed by flexure shear failure that’s lead to brittle failure. The effect of surface preparation is illustrated in “Figure 7 f”. The mode of failure changed from FRP rupture (for specimen with putty filler layer) to the plate end debonding (for specimen without surface preparation). The flexural capacity was improved by 10% by adding a putty filler layer as a base layer for GFRP sheet.
Figure 7 Load verses mid span deflection curves for test specimens

CONCLUSIONS

It was determined that one or two strips of GFRP (200 mm wide) (8 in.) attached on the exterior surface of the reinforced wall improves the out-of-plane strength significantly by (230 to 300 %) compared to the control specimen. The strength was increased by about (40 to 80 %) when one or two strips of the CFRP laminate were used. This improvement in capacity is equivalent to (3-5) g of acceleration and the deflection at the order of L/30. In terms of strength and ductility, the specimens with GFRP displayed better behavior than the specimens with CFRP. This behavior is due to the existing of silica fume and epoxy as a filler layer and the high debonding strain for epoxy compared to SikaDur30 in addition to high stiffness of CFRP. Also the existing of this layer improved the flexural capacity and changed the mode of failure compare with the same strengthened specimen without filler layer. The stiffness of the strengthened wall depend on external FRP and the internal steel reinforcement, while the ultimate load capacity and post peak behavior depend on controlling mode of failure which is independent of the steel reinforcement ratio. The test results indicated that the flexural behavior of reinforced masonry walls strengthened externally by FRP may be controlled by FRP fracture, debonding (intermediate crack or plate end debonding failure), or compressive concrete crushing failure.

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REFERENCES

EXTERNAL STRENGTHENING OF MASONRY SEMICIRCULAR ARCHES BY MONOLAYER PREPREG COMPOSITES

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ABSTRACT

An increasing research trend is to improve the structural performance of existing structures with new generation high performance materials. Among those, advanced composite materials have been significantly used as means of innovative solutions to strengthen existing structures. The most common usage of these materials is the external strengthening applications on structural components. The main objective of this study is to investigate the behaviour of prepreg strengthened semicircular masonry arches, and to highlight the effect of strengthening configurations on the structural performance of the arches. For this purpose, a series of masonry brick arches were prepared and strengthened by using unidirectional monolayer prepreg composites, which were produced in a prepreg machine. In the study, three different strengthening configurations were investigated, namely the Continuous Strengthening at the Intrados surface (CSI), Continuous Strengthening at the Extrados surface (CSE) and Localized Strengthening at the Intrados and Extrados surfaces (LSIE). The findings show that the monolayer prepreg composites play an important role in the strengthening of the masonry arches and the composites significantly enhance the load carrying capacity of the arches.

KEYWORDS

Composite materials, experiments, masonry arches, monolayer prepreg composites.

INTRODUCTION

Masonry materials have been used in the construction industry for centuries and masonry structures still represent a considerable portion of the building stock in the world. Although many masonry structures are currently kept in service, the majority of these structures are not quite able to resist large seismic forces. Therefore, development of effective strengthening methods for these existing structures is important for resilient communities and the preservation of these structures is still challenging in the structural engineering community. Restoration, strengthening and reinforcement of masonry structures have recently seen heightened interest in structural engineering. As a response to this challenge, using composites as strengthening materials can be made very effective. There is strong evidence that such materials have been used in different applications lately. In the last few decades, several composite materials have come to the forefront of strengthening due to their proven effectiveness for the structural and seismic retrofitting of existing structures. There are several research studies, which highlighted the common use of composite materials in different applications (Staab, 2015; Wang et al., 2011; Ye, 2003). A major portion of the studies focused on carbon fibre reinforced polymer (CFRP) and glass fibre reinforced polymer (GFRP) composites and their contributions to the structural performance (Grande and Milani, 2016; Mazzotti et al., 2015; Reboul and Ferrier, 2015; Milani and Lourenço, 2013; Lorenzis et al., 2013; Fedele and Milani, 2012; Witzany et al., 2011; Mosalam et al., 2007; Mosallam and Mosalam, 2003). Moreover, a considerable portion of the past research focused on the use of CFRP and GFRP composites in the masonry arches (Pintucchi and Zani, 2016; Castori et al., 2016; Caporale et al., 2014; Borri et al., 2011; Cancelliere et al., 2010; Lorenzis et al., 2007).

Although significant research has been conducted on strengthening masonry components with CFRP and GFRP composites, the application of prepreg composites for strengthening of masonry structures is relatively limited.
Therefore, this study focuses on the monolayer prepreg composites and their effects on the structural behaviour of the masonry arches. In the scope of this study, three different strengthening configurations, namely Continuous Strengthening at the Intrados surface (CSI), Continuous Strengthening at the Extrados surface (CSE), and Localized Strengthening at the Intrados and Extrados surfaces (LSIE), are experimentally investigated.

**MATERIALS AND METHODS**

**Manufacturing of Monolayer Prepreg Composite**

Prepreg is short for “pre-impregnated” fibre-reinforced composites. Prepreg composite is a multi-phase material that is composed of resin matrix and fibre reinforcement. In this study, the used resin matrix was composed of Araldite®LY 1564 SP epoxy resin and Aradur®XB 3486 hardener, which were weightily mixed in the ratio of 100:34, respectively. Furthermore, 12 K A-42 carbon fibre was used as a reinforcement material and the carbon fibres were supplied by DowAksa Advanced Composites Holdings BV (Turkey). In the production phase, all composites were manufactured in a unidirectional monolayer form (one direction of reinforcement) by using a special prepreg machine (Figure 1).

![Prepreg machine components](image1.png)

**Figure 1 Prepreg machine components**

After production of the composites, the monolayer prepreg composites were observed using Scanning Electron Microscopy (SEM) technology (Figure 2). SEM observations were conducted with a JEOL 6510-LV JSM SEM equipment of the laboratories of Marmara Research Centre of the Scientific and Technological Research Council of Turkey (TUBITAK MAM).

![SEM images of the prepreg composite](image2.png)

**Figure 2 SEM images of the prepreg composite**

**Preparation of Lime-based Mortar**

Lime-based mortar was produced by six principal components: hydraulic lime, stone and brick powder, fine sand, lime paste and water, and cement-free natural hydraulic lime (Albaria Calce Albazzana®) obtained from BASF Chemical Company and used as a binder material. Other components were locally selected for the mortar. The components were mixed in equal proportion by weight to prepare the lime-based mortar. To obtain good quality mortar, the mixture was blended by Steel Drum Mortar Mixer.
Preparation of the Masonry Arches

In the experiments, the arches were built of 29 standard hollow bricks, which had sizes of 90×190×50 mm (Figure 3). The clay bricks were obtained from a local brick company in Erzurum, Turkey. In the first step, the bricks were cleaned with water to remove any contamination and dust from the surfaces, and to enable an effective adherence between the brick and the mortar. After surface cleaning, masonry bricks were dried at room temperature for two days. The arches were constructed on a wood scaffolding with the desired geometrical properties. The internal diameter, internal rise and thickness of the arches were 1000 mm, 500 mm and 90 mm, respectively. The joints between the bricks were filled with the prepared lime-based mortar and the mortar joints were kept with a constant thickness of almost 5 mm.

CHARACTERIZATION OF THE MATERIALS

Mechanical Properties of the Composite

The tensile strengths of the prepreg composites were assessed by taking five representative specimens having dimensions of 25×250 mm. ASTM D 3039 standard was followed for the determination of the tensile strength of the prepreg composite using a Shimadzu universal tensile testing machine. All tensile tests were conducted considering the same conditions. The tensile strengths of the prepreg composites are given in Table 1.

<table>
<thead>
<tr>
<th>Samples</th>
<th>Tensile Strength (N/mm²)</th>
<th>Density (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>76.04</td>
<td>1421</td>
</tr>
<tr>
<td>2</td>
<td>75.92</td>
<td>1426</td>
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<tr>
<td>3</td>
<td>76.02</td>
<td>1415</td>
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<td>4</td>
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<td>1420</td>
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<tr>
<td>5</td>
<td>76.10</td>
<td>1419</td>
</tr>
<tr>
<td>Average</td>
<td>76.01</td>
<td>1420</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>0.70</td>
<td>3.96</td>
</tr>
</tbody>
</table>

Mechanical Properties of the Masonry Materials

Masonry materials used in the arches were tested to determine their mechanical characteristics. Therefore, clay bricks and mortar samples were tested in compression and three-point bending (Figure 4). These tests were performed based on the Turkish Building Codes (TS 699; TS EN 1467; TS EN 1469). In the mechanical tests, mortar samples at 7 and 28 days were used in order to obtain the mechanical properties (Tables 2 and 3).
Table 2 Mechanical properties of the bricks

<table>
<thead>
<tr>
<th>Samples</th>
<th>Compressive Strength (N/mm²)</th>
<th>Tensile Strength (N/mm²)</th>
<th>Density (kg/m³)</th>
</tr>
</thead>
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<tr>
<td>1</td>
<td>16.00</td>
<td>1.97</td>
<td>2861</td>
</tr>
<tr>
<td>2</td>
<td>16.01</td>
<td>1.99</td>
<td>2859</td>
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<tr>
<td>3</td>
<td>16.08</td>
<td>2.06</td>
<td>2872</td>
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<tr>
<td>4</td>
<td>15.98</td>
<td>1.95</td>
<td>2866</td>
</tr>
<tr>
<td>5</td>
<td>16.07</td>
<td>2.05</td>
<td>2862</td>
</tr>
<tr>
<td>Average</td>
<td>16.03</td>
<td>2.00</td>
<td>2864</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>0.45</td>
<td>0.82</td>
<td>5.15</td>
</tr>
</tbody>
</table>

Table 3 Mechanical properties of the mortar

<table>
<thead>
<tr>
<th>Samples</th>
<th>Compressive Strength (kg/cm²)</th>
<th>Tensile Strength (kg/cm²)</th>
<th>Density (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7 days</td>
<td>10.44</td>
<td>1.11</td>
<td>2125</td>
</tr>
<tr>
<td>28 days</td>
<td>11.39</td>
<td>1.49</td>
<td>2141</td>
</tr>
<tr>
<td>7 days</td>
<td>11.31</td>
<td>0.98</td>
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<td>28 days</td>
<td>11.26</td>
<td>1.53</td>
<td>2110</td>
</tr>
<tr>
<td>7 days</td>
<td>11.69</td>
<td>1.00</td>
<td>2166</td>
</tr>
<tr>
<td>28 days</td>
<td>11.22</td>
<td>1.54</td>
<td>2131</td>
</tr>
<tr>
<td>Average</td>
<td>11.22</td>
<td>1.53</td>
<td>2131</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>4.77</td>
<td>0.53</td>
<td>23.25</td>
</tr>
</tbody>
</table>

Strengthening Procedure

All masonry arches were strengthened using three different combinations, i.e. CSI, CSE, and LSIE. Before the application of the prepreg composites, the rough surfaces of the masonry arches were grinded and cleaned in order to remove any contaminations. Subsequently, the adhesive material used between the composites and arch surfaces was prepared by using Araldite AV 138 M-1 epoxy and HV 998 hardener and applied on the arch surface as a putty layer using a spatula (Figure 5). Finally, the prepreg composites were carefully applied to the arch surfaces and the strengthened arches were cured for ten weeks. The strengthening configurations investigated in this study are schematically presented in Figure 6.
TESTING PROCEDURE AND OBSERVATIONS OF THE MASONRY ARCHES

A series of experimental investigations were conducted to determine the maximum load bearing capacity and failure mechanisms. The arches were tested while rigidly attached in a reinforced concrete rigid platform. In the experimental study, each of the arches was subjected to gradually increasing concentrated vertical force applied at the mid-span until failure. The load was applied by a hydraulic jack and it was measured by a 250 kN capacity HBM load cell. The horizontal and vertical displacements were acquired by seven displacement transducers (LVDTs). Furthermore, five strain gauges (STRG) were used to measure the strains of the prepreg composites. The test setup and position of the loading and measuring devices are shown in Figure 7.

Unstrengthened Arch

The unstrengthened arch was first tested in the experimental study. During the test, the first hinge took place at the keystone when external load was 1.5 kN. After the load reached 2 kN, the second hinge occurred between the 7th and 8th bricks, then the unstrengthened arch failed when the load of the arch reached $F = 2.39$ kN. The unstrengthened arch experienced brittle failure due to formation of a three-hinge mechanism and all separations occurred in the interfaces between mortar joints and the bricks (Figure 8). Figure 9 shows the load-displacement curve corresponding to the loading point and the energy absorption capacity of the unstrengthened arch.
Continuously Strengthened at the Intrados surface (CSI)

In the CSI arch test, when the load reached 5.5 kN, debonding mechanism was observed at the prepreg-masonry arch interface. Subsequently, the eight bricks, numbered 14 to 21 became vulnerable and the first crack initiated between the 15th and 16th bricks. As a result, the ultimate load value of the CSI arch was 6.37 kN. Figure 10 shows the failure mechanism of the CSI arch. Figure 11 shows the load-displacement curve corresponding to the loading point and the energy absorption capacity of the CSI arch.

Continuously Strengthened at the Extrados surface (CSE)

The first debonding of the prepreg composite appeared during testing the CSE arch after reaching the load value of 5.20 kN. The prepreg was totally debonded between the 1st and 5th bricks, when the loading reached 9 kN.
first crack appeared between the 4th and 5th bricks at a load of 10.71 kN. As for the failure mode, the CSE arch collapsed due to the separation between the 14th and 15th bricks (Figure 12). Figure 13 shows the load-displacement curve corresponding to the loading point and the energy absorption capacity of the CSE arch.

![Figure 12 The failure mechanism of the CSE arch](image1)

Figure 12 The failure mechanism of the CSE arch

![Figure 13 CSE test results (a) Load-displacement curve, (b) Energy absorption capacity](image2)

Figure 13 CSE test results (a) Load-displacement curve, (b) Energy absorption capacity

Locally Strengthened at the Intrados and Extrados surface (LSIE)

In the LSIE arch test, the first crack became visible between the 4th and 5th bricks at the load of 1.5 kN. Next crack appeared between the 11th and 12th bricks at the external load of 2 kN. At a load of 2.5 kN, the third crack appeared in the arch. Finally, the LSIE arch failed in the mode of three-hinged collapse mechanism with an ultimate load of 4.56 kN. The collapse mechanism is presented in Figure 14. Figure 15 shows the load-displacement curve corresponding to the loading point and the energy absorption capacity of the LSIE arch.

DISCUSSION OF RESULTS

The main focus of this study was to examine the semicircular masonry arches, which were constructed using bricks and hydraulic lime-based mortar. They were also strengthened using monolayer prepreg composites with three different strengthening configurations on the extrados and intrados surfaces of the arches. In the study, the arches were loaded by incremental load at the mid-span and the structural behaviour of the arches and the prepreg composites were investigated from the experimental measurements. The experimental results showed that the load carrying capacity of the unstrengthened arch was 2.39 kN. The strengthened CSI arch had higher load carrying capacity as 6.37 kN. For the strengthened CSE and LSIE arches, the load carrying capacity reached values of 10.71 kN and 4.56 kN, respectively (Figure 16). Thus, the prepreg composites clearly contributed to the load carrying capacity, which compared to the load carrying capacity of the unstrengthened arch, was 266% for the CSI arch, 448% for the CSE arch, and 190% for the LSIE arch.
When the displacements of the arches were examined, the maximum displacement was calculated in the unstrengthened arch that was about 50 mm at the quarter point of the span. With respect to the strengthened arches, the maximum displacements were measured in the range of 10 mm and 15 mm at mid-spans. When the results of the energy absorption capacity of the arches were examined, the CSE arch had the maximum energy absorption capacity and it was almost two times higher than that of the unstrengthened arch (Figure 17).
When the results of the strain gauges on the prepreg composites were examined, small strains of the prepreg composites were observed. During the experimental study, the prepreg composites frequently debonded from the arch surfaces. Therefore, the prepreg did not provide the desired contribution as explained by the low values of the measured strains on the prepreg composites (Figure 18). The discrepancy for STRG3 (CSI) and STRG2 and STRG4 (CSE) in Figure 18 is attributed to debonding of the composite material at very early stages of loading.

**CONCLUSIONS**

Composite materials continue to remain popular in various engineering applications, including providing innovative and efficient solutions as strengthening materials for the construction industry. Considering easy installation, low weight, and high strength, composite materials have been used as strengthening materials for many existing structures. Therefore, strengthening of existing structures with composite materials has been a popular research area. This study mainly focused on masonry brick arches and investigated the structural strengthening of the arches with monolayer prepreg composites.

The examination of the experimental results showed that the monolayer prepreg composites have a considerable effect on enhancing the load carrying capacity and other structural properties of the arches. The prepreg composites clearly contributed to the load carrying capacity, which compared to the load carrying capacity of the unstrengthened arch, was 266% for the CSI arch, 448% for the CSE arch, and 190% for the LSIE arch. Moreover, the experimental study showed that the prepreg composites generally improve the energy absorption capacity of the brick masonry arches. When the results of the energy absorption capacity of the arches were examined, the energy absorption capacity of the CSI and CSE arches enhanced 130% and 194%, respectively. In addition, the structural interaction at the interfaces between the masonry arches and the prepreg composites plays an important role in the structural behaviour and the effectiveness of the strengthening of the arches. Therefore, the successful application of the prepreg composites heavily relies on better understanding of the debonding mechanism. It is emphasized that the monolayer prepreg composites provide excellent and versatile solutions for strengthening masonry structures.
ACKNOWLEDGMENTS

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REFERENCES

INVESTIGATION OF THE SHEAR BEHAVIOUR OF CONCRETE SLABS STRENGTHENED WITH ETS FRP BARS

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ABSTRACT

Reinforced concrete bridge deck slabs without shear reinforcement need to resist high magnitude concentrated loads. The governing failure mechanism of reinforced concrete bridge deck slabs is commonly shear, as it has been shown in experimental tests in the literatures. However, the existing shear strengthening methods, such as Externally Bonded (EB) FRP technique and Near Surface Mounted (NSM) FRP technique, are unavailable to apply in the shear damaged concrete bridge deck slabs. Embedded Through-Section (ETS) strengthening method is a relatively recent shear strengthening strategy for reinforced concrete components and consists of the opening holes across the structural thickness, where FRP bars are introduced and bonded to the concrete substrate with adhesive materials. Therefore, the ETS strengthening method could be suitable to improve the shear behaviour of concrete bridge deck slabs. To assess the effectiveness of this technique used in concrete slabs, a comprehensive experimental programme was carried out. In this test, the types of embedded materials and spacing of embedded rods were varied to investigate the behaviour of ETS FRP bar strengthening concrete slabs. The test results illustrated that the failure mode of concrete slabs were varied from shear failure to flexural failure by the application of ETS strengthening method and the ultimate strengths were also enhanced by this strengthening strategy. The results of this works show that the ETS strengthening scheme is effective and provides significant improvement in the shear behaviour of concrete slabs.

KEYWORDS

FRP, ETS strengthening, concrete slabs, shear behaviour, experimental test.

INTRODUCTION

Reinforced concrete slabs subjected to concentrated loads near linear supports are commonly found in practice, such as bridge deck slabs (Natário et al. 2014). The structural elements are characterized by high shear forces concentrated in the region between the concentrated loads and the linear support. Due to the increasing traffic loads and heavy truck load close to supports, the existing bridge deck slabs could fail in shear. Thus, more massive construction or shear reinforcement are now required in bridge deck slabs, whereas this formerly the case (Rombach and Latte 2008). To assess the question whether the lack of safety for existing bridge deck slabs that have been built mainly without shear reinforcement, or whether deck slabs under concentrated wheel loads exhibit reserves of shear capacity which have been neglected in current design codes (Rombach and Kohl 2013).

Currently, the shear strengthening techniques based on the use of fibre reinforced polymer (FRP) materials have been proposed and developed in the past thirty years (Teng et al. 2003; Lorenzis and Teng 2007). Externally bonded (EB) FRP is the most commonly used method for strengthening concrete structures using FRP material. To increase the shear behaviour of reinforced concrete structures, FRP sheets are generally applied on the side surface of concrete elements. Additionally, the near-surface-mounted (NSM) FRP rod method is another technique used to increase the shear resistance. In the NSM method, FRP rods were embedded grooves intentionally prepared on the concrete cover of the side faces of concrete structures. The efficiency of these strengthening schemes relies on the bond performance of concrete-adhesive-FRP interfaces. However, those two strengthening methods cannot be applied to increase the shear capacity of concrete deck slabs, due to inaccessible web of structures. Therefore, a
new strengthening approach is adopted (see Figure 1): vertical holes are drilled into the deck slabs upwards from the soffit in the shear zones, high-viscosity epoxy resin is injected and then FRP bars are embedded in to place. This strengthening method is called deep embedment strengthening (Valerio et al. 2009) or embedded through-section (ETS) strengthening (Mofidi et al. 2012). The research works in the literatures (Valerio et al. 2005; Chaallalet al. 2011; Barros et al. 2008) revealed that this strengthening technique provided higher strengthening efficiency compared to the EB and NSM strengthening method. In addition, the shear capacity of strengthened concrete beams was enhanced by this strengthening method.

![Figure 1 ETS strengthening process](image1)

The objective of this research study is to investigate the effect of the following variables on concrete slabs strengthened in shear using the ETS methods: (1) material type of strengthening materials; (2) spacing of FRP rods; (3) FRP rod diameters; (4) Damage of drilling holes. In addition, an experimental campaign of pull-out tests on carbon, glass, basalt and steel bars embedded into concrete is also presented, which defines the parameters for the design of strengthening scheme.

**EXPERIMENTAL PROGRAM**

**Bond Test**

![Figure 2 Test specimens for bond test](image2)

![Figure 3 Bond-slip behaviour of ETS strengthened test specimens](image3)

To assess the bond properties and optimise the choice of the strengthening rods for the ETS technique for concrete slabs, a series of pull-out tests on different strengthening rods epoxy-resined into 150x150x150mm concrete cubes were performed (see Figure 2). In this study, four types of strengthening materials, such as CFRP, GFRP, BFRP and steel, were adopted to investigate the bonding-slip behaviour of strengthening rods embedded through concrete
materials. As shown in Table 1, the failure mode of steel embedded specimens was debonding failure. On the other hand, all the test specimens embedded by FRP rods failed in concrete cracking, see Table 1. This could be due to the better compatibility between FRP and concrete compared to steel. In addition, the bond test results indicated that the bond strength and bond-slip behaviour could not be affected by the variation of strengthening materials, see Figure 3. The mechanical properties of embedded bar and adhesive were shown in Table 2.

<table>
<thead>
<tr>
<th>Strengthening material</th>
<th>Bond length (mm)</th>
<th>Diameter (mm)</th>
<th>Failure mode</th>
<th>Bond Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>steel</td>
<td>150</td>
<td>10</td>
<td>Debonding and yielding</td>
<td>7.8</td>
</tr>
<tr>
<td>BFRP</td>
<td>150</td>
<td>9</td>
<td>Concrete cracking</td>
<td>7.4</td>
</tr>
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<td>GFRP</td>
<td>150</td>
<td>9</td>
<td>Concrete cracking</td>
<td>7.7</td>
</tr>
<tr>
<td>CFRP</td>
<td>150</td>
<td>9</td>
<td>Concrete cracking</td>
<td>7.7</td>
</tr>
</tbody>
</table>

Table 2 Mechanical property of embedded bar and adhesives

<table>
<thead>
<tr>
<th>ETS Bar Type / Epoxy</th>
<th>Diameter (mm)</th>
<th>Tensile Strength (N/mm²)</th>
<th>Elastic modulus (N/mm²)</th>
</tr>
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<tbody>
<tr>
<td>Steel</td>
<td>10</td>
<td>504</td>
<td>200000</td>
</tr>
<tr>
<td>CFRP</td>
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<td>1581</td>
<td>156000</td>
</tr>
<tr>
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<td>1011</td>
<td>83000</td>
</tr>
<tr>
<td>GFRP</td>
<td>9</td>
<td>835</td>
<td>48000</td>
</tr>
</tbody>
</table>

Slab test programme

The experimental programme (see Table 3) involves eight tests performed on full-scale concrete slabs. Figure 4 presents the geometry and the reinforcement arrangement of the eight concrete slabs of the experimental programme. The reinforcement percentage and loading position (see Figure 4) was designed according to the current design code, which was adopted to force the occurrence of shear failure mode for the unstrengthened concrete slabs under a three-point load configuration. As shown in Table 3, some structural parameters were varied to investigate the shear strengthening effect of ETS scheme for concrete slabs, which included the strengthening materials, spacing of FRP strengthening rods and diameter of FRP strengthening rods. In addition, a test specimen was drilled but no strengthened was conducted to investigate the damage of drilling process.

Figure 4 Dimension of strengthened test specimens

The experimental programme (see Table 3) involves eight tests performed on full-scale concrete slabs. Figure 4 presents the geometry and the reinforcement arrangement of the eight concrete slabs of the experimental programme. The reinforcement percentage and loading position (see Figure 4) was designed according to the current design code, which was adopted to force the occurrence of shear failure mode for the unstrengthened concrete slabs under a three-point load configuration. As shown in Table 3, some structural parameters were varied to investigate the shear strengthening effect of ETS scheme for concrete slabs, which included the strengthening materials, spacing of FRP strengthening rods and diameter of FRP strengthening rods. In addition, a test specimen was drilled but no strengthened was conducted to investigate the damage of drilling process.
The vertical displacement was measured at the position under the applied load using linear variable displacement transducers (LVDTs) as shown in Figure 5a. The strain values of strengthening rods and steel reinforcement was measured by the electronic strain gauges and the locations of those strain gauges were shown in Figure 5b. The strain values of concrete slabs through the depth were measured by the vibrating-wire strain gauges, see Figure 5c.

**EXPERIMENTAL TEST RESULT**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cracking load (kN)</th>
<th>Failure load (kN)</th>
<th>Loading point deflection (mm)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-con</td>
<td>30</td>
<td>130</td>
<td>8.67</td>
<td>shear</td>
</tr>
<tr>
<td>BD1-16</td>
<td>35</td>
<td>134</td>
<td>8.34</td>
<td>shear</td>
</tr>
<tr>
<td>BS2-10</td>
<td>30</td>
<td>142</td>
<td>22.54</td>
<td>flexure</td>
</tr>
<tr>
<td>BB2-9</td>
<td>30</td>
<td>144</td>
<td>17.61</td>
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<tr>
<td>BG2-9</td>
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<td>140</td>
<td>19.36</td>
<td>flexure</td>
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<td>BC2-9</td>
<td>25</td>
<td>142</td>
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<tr>
<td>BC1-9</td>
<td>30</td>
<td>142</td>
<td>18.7</td>
<td>flexure-shear</td>
</tr>
<tr>
<td>BC1-13</td>
<td>30</td>
<td>148</td>
<td>20.58</td>
<td>flexure</td>
</tr>
</tbody>
</table>

Table 4 shows the cracking loads, the loading-carrying capacity, the vertical deflection under the loading point at failure and the failure mode of all the test slabs. All the test slabs cracked in flexure around 30kN. Those test slabs strengthened in shear with steel or FRP bars in shear span attained full ductile flexural response. The unstrengthened test slabs failed in brittle shear, see Table 4 and Figure 6. As shown in Figure 6, the shear discontinuity was constrained to occur between the ETS bars and loading point. Therefore, the loading capacity was enhanced by around 10% with the ETS strengthening method, as seen in Table 4. However, the increasing degree of this test slabs was smaller than that obtained in the test results of the ETS strengthened beams (Valerio et al.2005), because the flexural capacity of the unstrengthened test slab is a bit larger than shear capacity of that specimens. In addition, no debonding failure occurred in all the strengthened slabs, which is attributed to the increased bonding effect of ETS method. In the comparison of test results from models coded as B-con and BD1-16, it was found that the drilling process would influence the structural behaviour of concrete slabs. The test results indicated that the material type of embedded bars could not influence the behaviour of strengthened test slabs significantly, which is attributed to the good bond behaviour of ETS bars and concrete. In addition, it was shown in the test that failure mode of test slabs was varied from flexural failure to shear-flexural failure by reducing the
number of embedded bars. Interestingly, increasing the diameter of embedded bars resulted in a bit larger ultimate capacity and vertical deflections.

![Images](a) B-con  (b) BC2-9  
(c) BC1-9  (d) BC1-13

Figure 6 Cracking patterns of test specimens

Figure 7 shows the load-deflection at loading point for all the test specimens, clearly demonstrating the flexural ductility that exhibited those specimens containing sufficient shear strengthening. To investigate the effect of ETS strengthening method on the ductility of concrete slabs, a deformability factor (DF) (Vijay and GangaRao 2001) defined as the ratio of energy absorption at ultimate (area under load-deflection curve up to the failure load) to the energy absorption at service load (at serviceability deflection limit of span/250) was used in this paper. As shown in Table 5, the ductility of concrete slabs was enhanced by more than 200% in the comparison between unstrengthened and strengthened slabs. The ductility of strengthened concrete slabs could be enhanced by increasing the stiffness of ETS bars.

![Figure 7 Load vs vertical deflection at loading point](a) B-con  (b) BC2-9  
(c) BC1-9  (d) BC1-13

**Figure 7 Load vs vertical deflection at loading point**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>DF*</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-con</td>
<td>1.05</td>
</tr>
<tr>
<td>BD1-16</td>
<td>1</td>
</tr>
<tr>
<td>BS2-10</td>
<td>4.06</td>
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<tr>
<td>BB2-9</td>
<td>3.14</td>
</tr>
<tr>
<td>BG2-9</td>
<td>3.39</td>
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<tr>
<td>BC2-9</td>
<td>2.99</td>
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<tr>
<td>BC1-9</td>
<td>3.45</td>
</tr>
<tr>
<td>BC1-13</td>
<td>3.72</td>
</tr>
</tbody>
</table>

*DF= deformability factor defined by Vijay and GangaRao 2001;
CONCLUSION

This study presents the relevant results of experimental programme for assessment of the effectiveness of the ETS FRP technique for the shear strengthening of reinforced concrete slabs. The following conclusions can be drawn from this experimental investigation.

(1) The system effectiveness relies on the bond between the ETS bars and the concrete. A series of pull-out tests has shown that the bond-slip response of the system is robust and ductile. Due to the good compatibility between FRP and concrete, no debonding failure occurred in the test specimen embedded by FRP bars.

(2) No major differences were noticed between the slabs strengthened with steel bars and those strengthened with FRP bars, but the use of FRP is suggested for such strengthening, due to its lightness and corrosion resistance.

(3) Due to the loading location close to the support, shear failure occurred in the unstrengthened test slabs. The failure mode is brittle and sudden. The ETS shear strengthening technique can be used to avoid the occurrence of shear failure of concrete slabs subjected to the load close to supports. It was found that failure mode of concrete slabs were varied from brittle shear failure to ductile flexural failure. It is due to the continuity of shear cracking development was broken by the ETS strengthening method.

(4) In this study, it was found that the material type of ETS strengthening materials cannot influence the behaviour of strengthened test slabs. Interestingly, increasing the diameter of ETS FRP bars resulted in larger ultimate capacity and higher ductility.

(5) Due to the small flexural stiffness of test slabs, the ultimate capacity was enhanced by around 10%. However, the ductility of test slabs was improved significantly. The vertical deflection at the failure was increased by more than 100% and the ductility was increased by more than 200%.

ACKNOWLEDGMENTS

The authors wish to express their sincere appreciation of the Foundation of Guangdong Distinguished Young Scholar Planning (Yq2013155), the National Science Natural Science Foundation of China (50908055, 51308117) and Division of Transportation Guangdong Province (2013-02-029) in supporting this research.

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EFFECT OF SHEAR SPAN-DEPTH RATIO ON BEHAVIOR OF RC BEAMS SHEAR-STRENGTHENED WITH VARIOUS FRP WRAPPING CONFIGURATIONS

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ABSTRACT

The shear span/depth ratio is known to affect the behavior of reinforced concrete (RC) beams. However, for RC beams shear-strengthened with fiber-reinforced polymer (FRP), due to insufficient understanding of its effect, the shear span/depth ratio is not considered in existing design guidelines. This paper focuses on the effect of shear span/depth ratio on the behavior of RC beams shear-strengthened in full-wrapping and U-wrapping CFRP configurations. In total, twelve strengthened beams and six normal beams are tested with shear span/depth ratios from 1.0 to 3.5. The experimental results indicate that the shear span/depth ratio can significantly affect the shear behavior of strengthened RC beams, but in different ways for the two FRP strengthening configurations, especially in the FRP shear contribution and distribution of FRP strain along a critical shear crack. Furthermore, the authors’ experimental results, as well as test results from the literature, are compared to predictions from various design guidelines. For both full-wrapping and U-wrapping configurations, the results show the same trend for our tests and those from the literature.

KEYWORDS

Fiber-reinforced polymer (FRP), reinforced concrete (RC) beam, shear-strengthening, shear span/depth ratio, design guideline.

INTRODUCTION

Full-wrapping and U-wrapping fiber-reinforced polymer (FRP) configurations are two major shear-strengthening schemes in externally bonding (EB-) FRP composites for improving the shear resistance of deteriorated RC beams. In the literature, there are a lot of research results (e.g., Pellegrino and Modena 2002; Chen and Teng 2003a, b; Leung et al. 2007; Chen et al. 2010) showing that the shear strength of reinforced concrete (RC) beams can be substantially increased by full-wrapping or U-wrapping FRP composites working as external stirrups. Both experimental investigations and theoretical analyses have been carried out to explore various issues of RC beams strengthened in shear with EB-FRP. However, a comprehensive investigation covering a wide range of shear span-to-effective depth ratio (also called the shear span/depth ratio, or \(a_v/d\) ratio, where \(a_v\) and \(d\) are respectively the length of shear span and the effective length of a RC beam) is yet to be reported. Although shear span/depth ratio is a very important factor on the shear behavior of RC beams whether they are strengthened or not, only few investigations in the literature (e.g., Khalifa et al. 2000; Boussellam and Chaallal 2006) have tested shear-strengthened members with two or more shear span/depth ratios. Even in these experimental investigations, the effect of shear span/depth ratio was not the major focus. This paper will therefore focus on the effect of the shear span/depth ratio on the shear performances of RC beams strengthened with full-wrapping and U-wrapping CFRP strips. To study the strengthening effect for beams exhibiting various shear failure modes, a wide range of shear span/depth ratios from 1.0 to 3.5 at 0.5 intervals will be considered. After a description of the experimental program, the test results will be presented and discussed. Then, the effects of shear span/depth ratio for the two FRP configurations are compared. Furthermore, our own test results, as well as collected data from the literature, are compared to predictions from various design guidelines.

EXPERIMENTAL DESIGN

A total of eighteen RC beams were tested in the experimental investigation. Six normal RC beams, not strengthened with FRP, were used as control. Six RC members were shear-strengthened with U-wrapping CFRP strips while another six specimens were strengthened in shear with full-wrapping CFRP strips. The RC beams
were either 2,000 or 2,400 mm long. Shorter specimens were prepared for \(a/v > 2.5\), while longer beams were employed for \(a/v < 2.5\). All specimens were designed to fail in shear even after they have been strengthened in shear with CFRP strengthening configuration. The longitudinal steel reinforcement consisted of four reinforcing bars (diameter = 28 mm, yield strength = 450 MPa) at the bottom and two reinforcing bars (diameter = 32 mm, yield strength = 470 MPa) at the top. All specimens were tested under four point bending. One side of the beam (as the un-strengthened zone) was heavily reinforced in shear with stirrups (12-mm diameter at 85-mm spacing, yield strength = 390 MPa), whereas the other side (as the strengthened zone) was reinforced with 6.5-mm diameter stirrup (yield strength = 310 MPa) at 160-mm spacing. Experimental investigations would focus on the weaker side, which was expected to undergo shear failure even after strengthening with CFRP strips. In the un-strengthened zone, the shear span \(a_s\) is equal to \(a\) for beams with \(a/v \leq 2.0\); for beams with \(a/v > 2.5\), \(a_s\) will be fixed at 2\(d\). The CFRP strips were bonded discretely over the strengthened zone in a full-wrapping or U-wrapping strengthening configuration, where all strips were 60-mm wide and applied at 150-mm spacing (center-to-center). The effective thickness of CFRP layer was 0.11 mm. The tension strength of CFRP sheet is 4200 MPa, the tensile modulus is 235 GPa, and the elongation is 1.8%. The concrete for casting the beams was supplied by a local ready-mix producer. The shorter and longer beams were cast separately, and the average compressive strength from concrete cubes was 47 and 55 MPa, respectively. Strain gauges were glued on the longitudinal/transverse steel reinforcements and the concrete surface. To have good estimates of the effective strains of CFRP strips, which govern their resistance to shear failure, a series of strain gauges were installed on each CFRP strip to record, as accurately as possible, the strain level at the point intersected by the major shear crack. The load was applied by a hydraulic testing machine with the loading speed rate maintained at 0.01 mm/s. During testing, a data logger was used to record and store the test data, which were later transmitted to a computer for further processing.

**TEST RESULTS AND ANALYSIS**

**Overview of Experimental Observation**

In the four-point bending test, the twelve RC beams shear-strengthened with full-wrapping and U-wrapping CFRP strips were observed to fail in different shear failure modes with increasing the shear span/depth ratio, which can be classified into deep-beam failure, shear-compression failure, and shear-tension failure according to conventional reinforced concrete theory. The variation of shear failure mode with shear span/depth ratio is similar to that of the corresponding control specimens. Generally speaking, most of the FRP strips failed in debonding for U-wrapping FRP configuration, and in rupture for full-wrapping strengthening configuration. FRP strips not intersected by the critical shear crack (e.g., the strips close to the support) might not be damaged at the ultimate state of RC beams. Also, in the U-wrapping FRP configuration, a small number of strips close to the support might fail in complete/partial rupture as shear cracks intersect with the strip at a position near the bottom of the beam, where the FRP is effectively anchored through U-wrapping.

![Figure 1 Shear capacity and FRP shear contribution of all specimens](image_url)

**Shear Capacity and FRP Shear Contribution**

In previous experimental investigations, FRP strip were often considered as external stirrups in shear-strengthened beams (Khalifa et al. 2000; Triantafillou and Antonopoulos 2000), where the FRP shear contribution is obtained by subtracting the results for the control beam from the corresponding strengthened beam. Based on this approach,
the FRP shear contributions of all specimens are calculated and shown in Figure 1. For convenience of analysis, the corresponding strengthening percentage increments are also presented in Figure 1.

As shown in Figure 1, both U-wrapping and full-wrapping FRP strips can improve the shear resistance of RC beams, but the effectiveness varies with shear span/depth ratio. Furthermore, the full-wrapping configuration is more effective than the U-wrapping configuration for a given shear span/depth ratio. Although the improvements of shear capacities for the two FRP strengthening configurations are different, the trends of FRP shear contributions in both cases show a similar parabolic shape, increasing from the minimum value for the deep beam to a maximum value for a beam with medium span/depth ratio before decreasing with further increase in span/depth. This trend indicates that FRP shear contribution for both U-wrapping and full-wrapping configuration will vary while the shear failure mode changes with shear span/depth ratios.

The trend of FRP shear contribution with increasing shear span/depth ratio could be explained by considering three major aspects. First, as FRP strips play different roles in resisting shear under various shear failure modes, the FRP shear contribution varies with shear span/depth ratio. For a deep beam, the concrete arch carries most of the applied load and FRP strips act as hoops to restrict the deformation of the beam web in the vertical direction. FRP strips are hence not contributing much to the shear resistance. Second, the angle of the critical shear crack can affect the effectiveness of FRP strips in shear resistance. For a smaller shear span/depth ratio, the angle of the critical shear crack with respect to the fiber orientation is smaller, so the stress along the fiber direction is less effective in controlling the opening of the crack. Third, the FRP strips intersected by the critical shear crack will provide most of the FRP shear contribution, whereas the strips outside the critical shear crack should play an insignificant role in resisting shear. A similar explanation has been provided by Cao et al. (2005) and Li and Leung (2016).

**COMPARISON BETWEEN TEST RESULTS AND DIFFERENT DESIGN GUIDELINES**

The accurate prediction of the FRP shear contribution $V_f$ is a key issue for the development of design guidelines. In this paper, five existing design guidelines, i.e. ACI 440.2R (2008), FIB-TG 9.3 (2001), JSCE (2001), GB 50608 (2010), and CSA S806 (2012), are employed to obtain the FRP shear contribution for comparison with our own results, as well as the collected data from the literature. In the calculations, all safety factors will be taken out of the design equations because different design guidelines have different underlying theories and different ways of including safety factors. Apart from the safety factors, other limits on the strengthening effect will still be maintained.

**Comparison between Our Results and Predictions from Design Guidelines**

The experimental results (**abbr. Exp**) and predicting values (**abbr. Pre**) from the five design guidelines for the full-wrapping and U-wrapping configuration are shown in Figure 2. As shown in Figure 2, regardless of the design guideline adopted, the Pre/Exp ratio of FRP shear contribution $V_f$ is clearly dependent on the shear span/depth ratio, especially when $a_v/d \leq 1.5$ and $a_v/d \geq 3.0$. For both FRP configurations, the distribution trend of Pre/Exp ratios always decreases from the higher values for deep beams to the minimum values for medium shear spans before increasing with the shear span/depth ratio, but the curvatures of fitting lines are different for the two FRP configurations. For U-wrapping configuration, all the design guidelines severely overestimate the shear resistance of FRP strip for the deep beam. In the high shear span/depth ratios ($a_v/d \geq 3.0$), there is still a potential risk of overestimating the FRP shear contribution with the JSCE, FIB and GB design guidelines, where the Pre/Exp ratio is very close to or over 1.0. For full-wrapping FRP configuration, for $a_v/d = 1.0$, the FRP shear contributions predicted from two design guidelines, ACI and CSA, are conservative, whereas the other three design guidelines overestimate the experimental results. For shear span/depth ratio $a_v/d$ not less than 1.5, all the five design guidelines can predict conservatively the FRP shear contributions. However, the predicted $V_f$ from ACI and CSA guidelines seem to be overly conservative due to the individual limitations, such as limiting of effective FRP strain to 0.004 in ACI 440.2R and the limiting of FRP tensile strength to 0.005$E_f$ in CSA S806.

The above comparative analysis indicates that the predictions of the FRP shear contribution $V_f$ from existing design guidelines are affected strongly by the shear span/depth ratio, so the shear span/depth ratio shall be taken into account in design equations. Furthermore, there will be a potential risk in overestimating the FRP shear contribution for beams with low $a_v/d$ ratios (especially for deep beams) from existing design equations. Such a trend of the predicted FRP shear contribution from the five design guidelines can be explained by the following two major reasons. First, the shear strength models of these five design guidelines are all based on the traditional truss model, which is suitable for slender RC beams rather than for deep beams. As the diagonal tension reinforcement (which is provided by steel stirrups and FRP strips) has a relatively small effect on the concrete arch action for a deep beam, its contribution could be very low. Second, in these design guidelines (except for CSA
design guideline), the inclination angle of critical shear crack $\theta$ is assumed equal to 45°. Actually, the angle $\theta$ changes with the shear span/depth ratio, which is a function of section depth, reinforcement properties, and the internal forces (e.g., moment, shear and axial force) acting on it according to the modified compression field theory (MCFT) (Razaqpur and Spada 2015). Specifically, with changing shear span/depth ratio, the angle $\theta$ can be below or above 45°, which would lead to higher or lower prediction of the FRP shear contribution $V_f$.

**Comparison between Collected Data and Predictions from Design Guidelines**

To better understand the effect of shear span/depth ratio on the differences between predictions from existing design guidelines and collected data from the literature. Fifty-two test results ($1.50 \leq a_v/d \leq 3.80$) are chosen for U-wrapping configuration, while forty-three data ($1.43 \leq a_v/d \leq 3.00$) are chosen for the full-wrapping case.

For U-wrapping FRP configuration, the trend of the Pre/Exp ratio of $V_f$ between predictions from the five design guidelines and collected data shows high variations as FRP debonding failure is also sensitive to other factors such as member size, properties of materials and bond-slip behavior between FRP and concrete. For the FIB, JSCE and GB design guidelines, most of the Pre/Exp ratios are over 1.0 [Figures 3(b) - (d)]. The other two design guidelines often overestimate the shear resistance of FRP for $a_v/d \leq 2.0$ and $a_v/d \geq 3.00$ [Figures 3(a) and (e)]. For full-wrapping FRP configuration, as shown in Figure 4, both the ACI and CSA design guidelines can provide conservative predictions for most of the experimental data over the range of shear span/depth ratios considered. However, the predicted $V_f$ from these guidelines seem to be overly conservative especially in the range of $a_v/d$ ratio from 2.5 to 3.0. The predictions from the FIB, GB and JSCE guidelines overestimate the FRP shear contributions in many of the forty-three experimental results, especially for $a_v/d \leq 2.0$ and $a_v/d \geq 2.7$ [Figures 4(b) - (d)]. This clearly indicates a potential risk of overestimating the FRP shear contributions for both lower and higher $a_v/d$ ratios with these guidelines.

![Figure 2 Comparison of $V_f$ between our test results and predictions from design guidelines](image-url)
Figure 3 Comparison of $V_f$ between collected data and predictions from design guidelines (U-wrapping sheets)

Figure 4 Comparison of $V_f$ between collected data and predictions from design guidelines (full-wrapping sheets)
CONCLUSIONS

The experimental investigation in this thesis focuses on the effect of shear span/depth ratio on the mechanical behavior of RC beams shear strengthened with both U-wrapping and full-wrapping FRP configurations. The following conclusions can be drawn from the test results and experimental analysis:

1. EB-FRP strengthening technique can improve effectively the shear behavior of RC beams, especially the shear capacity. The improvement of shear capacity in the full-wrapping configuration is more significant than that in U-wrapping configuration.
2. The FRP shear contribution is significantly affected by the shear span/depth ratio not only in U-wrapping but also in full-wrapping configurations. The trend of FRP shear contribution shows a near-parabolic shape with increasing shear span/depth ratio from 1.0 to 3.5, which increases from the minimum value for a deep beam to the maximum value for the beam with medium span/depth ratio before decreasing with further increase in shear span/depth ratio.
3. For both FRP configurations, the predicted shear capacity from five existing design guidelines is all affected strongly by the shear span/depth ratio. In all five design guidelines, there is a potential risk in overestimating the FRP shear contribution for beams with lower a/d ratios (especially in deep beams).

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EXPERIMENTAL INVESTIGATION OF RC BEAMS STRENGTHENED IN SHEAR WITH EXTERNALLY BONDED COMPOSITES

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ABSTRACT
This paper presents the results of an experimental campaign carried out to investigate the behavior of reinforced concrete (RC) beams strengthened in shear using externally bonded advanced composite materials. In order to compare their performance, two different types of composite materials were used to strengthen the beams: fiber reinforced polymer (FRP) and fiber reinforced cementitious matrix (FRCM) composites. The beams were then tested in four-point bending scheme, and measurements regarding applied load and mid-span displacements were acquired. Observations regarding the gain in shear strength, influence on mid-span deflection and ductility, and comparison of the performance of the two strengthening systems are provided. For specimens strengthened with FRCM composite, the contribution to the shear strength provided by the FRCM strengthening system is compared with the value predicted by an analytical model found in the available literature.

KEYWORDS
FRP, FRCM, reinforced concrete, shear, strengthening.

INTRODUCTION
The growing necessity of strengthening existing reinforced concrete (RC) structures due to their deterioration with age or change in applied loads due to use modification or upgrading to currently available design codes has generated a great interest in researchers to develop time/cost efficient strengthening techniques. Among these techniques, Fiber Reinforced Polymer (FRP) composites have proven to be an adequate solution, providing additional strength for flexural, shear, and confinement applications while minimizing some disadvantages associated with traditional strengthening techniques such as increase in self weight of the structure, undesirable change in stiffness, and handling of heavy steel parts. However, some drawbacks of the use of FRP composites, linked mainly to the use of the epoxy resins, have been reported (Al-Salloum et al. 2012). For this reason, advanced composites in which the resin matrix is replaced by a cementitious matrix, known as Fiber Reinforced Cementitious Matrix (FRCM) composites, have recently raised interest in researchers worldwide. Although research performed on the topic is still scarce, the effectiveness of this technique for flexural and shear strengthening and confinement of axially/eccentrically loaded RC elements is confirmed by the available experimental evidence (e.g., Triantafillou et al. 2006; Bruckner et al. 2006, 2008; Blanksvard et al. 2009; Pellegrino and D’Antino 2013; and Ombres 2015).

This paper presents the preliminary results obtained from an on-going experimental investigation carried out at the University of Padua (Italy) on the behavior of RC beams strengthened in shear using externally bonded FRCM composites. The effectiveness of the FRCM composites in providing additional shear strength to RC beams is discussed and also compared with the effectiveness of FRP composites. Furthermore, the predicted contribution to the shear strength by the FRCM strengthening system computed by the model proposed by Triantafillou and Papanicolaou (2006) is compared to the experimental results.
EXPERIMENTAL PROGRAM

The experimental program carried out at the Construction Materials Testing Laboratory of the University of Padua (Italy) included the testing of six RC beams with the geometry and configuration of internal longitudinal and transverse reinforcement presented in Figure 1a. The longitudinal reinforcement ratio \( \rho_l = A_s / b w \) (\( A_s \)=longitudinal reinforcement area, \( b_w \)=beam width, \( d \)=beam effective depth) was 0.057. The beams were cast in two different batches. The average concrete compressive strength for the different batches, determined by testing 3 cylinders with dimensions \( \phi 100 \times 200 \) mm in accordance with EN 12390-3 (2009) within +/- 4 days of the day of testing, was 23.3 MPa and 21.3 MPa. The \( \phi 8 \) mm, \( \phi 16 \) mm, and \( \phi 26 \) mm deformed reinforcing steel bars had a measured yield strength (average of three specimens for each diameter) of 527 MPa, 535 MPa, and 545 MPa, respectively. The beams were tested using a four-point bending scheme with a total span length of 2.70 m, as shown in Figure 1. The shear span-to-depth ratio (\( a/d \)) was equal to 3.0. The test was controlled by slowly increasing the force. The loading was paused at different increments to mark cracks and take photographs.

The beams were designated according to the following convention:

\[ S1 - XXXX - F# - UY \]

where \( S1 \) corresponds to the stirrup spacing in the tested shear span (300 mm, center to center), \( XXXX \) defines the type of strengthening system (control, FRP, or FRCM), \( # \) is the type of fiber (1=carbon mesh, 3=carbon net, and 4=steel net), \( U \) indicates U-wrapped composite configuration, and \( Y \) is related to the use of composite anchors (N=no anchors, A=anchors). Table 1 presents the characteristics of the beams tested.

<table>
<thead>
<tr>
<th>Beam</th>
<th>( f'_c ) (MPa)</th>
<th>( b_w ) (mm)</th>
<th>( d ) (mm)</th>
<th>( a/d )</th>
<th>Composite Configuration</th>
<th>Anchors</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-CONTROL</td>
<td>23.3</td>
<td>150</td>
<td>250</td>
<td>3.0</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>S1-FRP-F1-UN</td>
<td>23.3</td>
<td>150</td>
<td>250</td>
<td>3.0</td>
<td>U-wrapped</td>
<td>No</td>
</tr>
<tr>
<td>S1-FRCM-F3-UN</td>
<td>23.3</td>
<td>150</td>
<td>250</td>
<td>3.0</td>
<td>U-wrapped</td>
<td>No</td>
</tr>
<tr>
<td>S1-FRCM-F3-UA</td>
<td>23.3</td>
<td>150</td>
<td>250</td>
<td>3.0</td>
<td>U-wrapped</td>
<td>Yes</td>
</tr>
<tr>
<td>S1-FRCM-F4-UN</td>
<td>21.3</td>
<td>150</td>
<td>250</td>
<td>3.0</td>
<td>U-wrapped</td>
<td>No</td>
</tr>
<tr>
<td>S1-FRCM-F4-UA</td>
<td>21.3</td>
<td>150</td>
<td>250</td>
<td>3.0</td>
<td>U-wrapped</td>
<td>Yes</td>
</tr>
</tbody>
</table>

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The beams were strengthened with a U-wrapped configuration with one layer of fibers. Before installing the FRP/FRCM system, the beam surface was subjected to mechanical grinding, the corners of the specimens were rounded to a radius of approximately 20 mm, and any loose sand grains were removed. For the FRCM system, the concrete surface was wetted and the first layer of cementitious matrix was immediately applied. The beams were strengthened in shear within just one of the shear spans, with a continuous strip (see Figure 1b). In order to force the shear failure in the designated span, the opposite shear span of the beam had steel stirrups spaced 75 mm (see Figure 1a).

The elastic modulus, ultimate tensile strength, overall area weight, and thickness of the fibers reported by the manufacturer are presented in Table 2 (G&PIntech 2016). For specimens strengthened with FRCM composite, the cementitious matrix was applied in layers of approximately 2 mm thickness. Three 40x40x160 mm samples taken from the cementitious matrix were tested +/- 4 days from the day of testing of the beams. The average flexural strength $f_{cf}$ (EN 12390-5 2009) and average compressive strength $f_{cm}$ (EN 12390-3 2009) were equal to 6.33 MPa (COV=0.15) and 45.2 MPa (CoV=0.039), respectively.

### Table 2 Composite fiber geometrical and mechanical properties (G&PIntech 2016)

<table>
<thead>
<tr>
<th>Fiber Type</th>
<th>Material</th>
<th>Overall Area Weight (g/m²)</th>
<th>Elastic Modulus (GPa)</th>
<th>Ultimate Tensile Strength (MPa)</th>
<th>Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1</td>
<td>Carbon</td>
<td>300</td>
<td>390</td>
<td>3000</td>
<td>0.084</td>
</tr>
<tr>
<td>F3</td>
<td>Carbon</td>
<td>170</td>
<td>240</td>
<td>4700</td>
<td>0.047*</td>
</tr>
<tr>
<td>F4</td>
<td>Steel</td>
<td>2200</td>
<td>190</td>
<td>2400</td>
<td>0.270*</td>
</tr>
</tbody>
</table>

*Equivalent nominal thickness (mm/m)

For beams S1-FRCM-F3-UA and S1-FRCM-F4-UA, spike anchors comprised of dry aramid fibers were used to in an attempt to anchor the composite to the beam side faces. The anchors were placed in holes drilled through the entire cross section of the beam located 100 mm from the top of the beam. After placing the anchors, the holes were filled with epoxy resin, and the anchors were fanned out and attached to the FRCM system using epoxy resin.

**RESULTS AND DISCUSSION**

Figure 2 shows the applied load (P) versus mid-span displacement ($\Delta$) curves for all specimens. The maximum load ($P_{max}$), the nominal shear strength ($V_n=0.5P_{max}$, neglecting the effects of self weight), the displacement at mid-span corresponding to $P_{max}$ ($\Delta P_{max}$), and the failure mode of each beam are summarized in Table 3. The additional shear strength provided by the strengthening system ($V_f$) and its efficiency relative to the nominal shear strength of the control beam ($V_f/V_{n,control}$) are also included in Table 3. $V_f$ is computed as the total shear strength of a given strengthened specimen minus the shear strength of the control specimen (S1-Control).

![Figure 2 Applied load (kN) vs. mid-span displacement (mm) curves for all specimens](image_url)
Table 3 Summary of tests results

<table>
<thead>
<tr>
<th>Beam</th>
<th>$P_{\text{max}}$ (kN)</th>
<th>$V_n$ (kN)</th>
<th>$V_f$ (kN)</th>
<th>$V_f/V_{n\text{-control}}$</th>
<th>$\Delta P_{\text{max}}$ (mm)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-CONTROL</td>
<td>230.5</td>
<td>115.2</td>
<td>-</td>
<td>-</td>
<td>15.4</td>
<td>Shear</td>
</tr>
<tr>
<td>S1-FRP-F1-UN</td>
<td>338.3</td>
<td>169.1</td>
<td>53.9</td>
<td>0.47</td>
<td>26.0</td>
<td>Flexure</td>
</tr>
<tr>
<td>S1-FRCM-F3-UN</td>
<td>284.8</td>
<td>142.4</td>
<td>27.2</td>
<td>0.24</td>
<td>17.8</td>
<td>Shear</td>
</tr>
<tr>
<td>S1-FRCM-F3-UA</td>
<td>290.3</td>
<td>145.1</td>
<td>29.9</td>
<td>0.26</td>
<td>19.8</td>
<td>Shear</td>
</tr>
<tr>
<td>S1-FRCM-F4-UN</td>
<td>299.5</td>
<td>149.7</td>
<td>34.5</td>
<td>0.30</td>
<td>17.7</td>
<td>Shear</td>
</tr>
<tr>
<td>S1-FRCM-F4-UA</td>
<td>300.3</td>
<td>150.1</td>
<td>34.9</td>
<td>0.30</td>
<td>20.6</td>
<td>Shear</td>
</tr>
</tbody>
</table>

The initial stiffness of the six beams was similar as shown in Figure 2. However, for the control beam, there is a reduction of the stiffness after a level of load associated with cracking of the concrete ($P=150$ kN) that is not noticeable in the other beams. Irrespective of the composite system (FRP, carbon-FRCM, or steel-FRCM), the curves for the strengthened specimens are similar up to failure, with a significant reduction of the stiffness close to $P_{\text{max}}$.

**Additional shear strength provided by the strengthening system**

From Table 3, it can be seen that using FRP composite the failure mode of S1-CONTROL was transformed from shear to flexure, with an increase in strength of $V_f/V_{n\text{-control}}=0.47$. All specimens strengthened with FRCM jackets failed in shear and had an increase in shear capacity ranging from 24% to 30%. Specimens with steel fibers had a slightly larger increase of $V_f$ than specimens with carbon fibers. In addition to the increase in shear strength, all strengthened beams had an increase of $\Delta P_{\text{max}}$ relative to the control beam.

The use of anchors resulted in a negligible increase of the shear strength for strengthened beams for both types of fibers, but allowed for attaining larger values of $\Delta P_{\text{max}}$ when compared with the corresponding beams without anchors. These observations are in agreement with the results of Baggio et al. (2014), who utilized anchors similar to those used in this study and witnessed a 3% increase in the strength when comparing FRCM strengthened beams with and without anchors. This behavior can be explained by the fact that some FRCM composites exhibit slippage of the fibers relative to the cementitious matrix during the debonding process (D’Antino et al. 2015; Tetta et al. 2015).

**Failure mode and cracking pattern**

Beam S1-CONTROL presented a typical beam shear failure characterized by the formation of a main diagonal crack as shown in Figure 3. It is important to notice that besides the main crack, minor diagonal cracks distributed along the shear span also formed. For beam S1-FRP-F1-UN, flexural failure caused by concrete crushing was observed. For this specimen, it was not possible to identify cracks in the strengthened shear span, although some shear cracks were recognized in the unstrengthened shear span of the beam.

In Figure 3, the cracking patterns for the beams strengthened with FRCM composites are also included. Unlike beam S1-FRP-F1-UN, cracking was visible on the surface of the FRCM jackets. This is an advantage of the FRCM system over the FRP system because it allows for immediate and easy inspection of damaged regions (Triantafillou and Papanicolaou 2006).
Beam S1-FRCM-F3-UN failed by diagonal tension. Local detachment of the entire thickness of the jacket close to the point of application of the load and fiber slippage along the main crack were observed. The inclination of the main diagonal crack was less steep than that of the control beam, and the distribution of the cracking was located in a smaller area. For beam S1-FRCM-F3-UA, the diagonal crack formed around the anchors, and composite detachment was precluded. Although there was not a significant increase in the shear strength, the width of the cracks was larger than those observed in the corresponding beam without anchors, which might explain the larger value of $\Delta P_{\text{max}}$ attained.

For beam S1-FRCM-F4-UN, failure was caused by detachment of the composite system. It is important to notice that the cracking pattern reflected in the steel FRCM jacket is different than those observed by the control specimen and the beams strengthened with carbon FRCM composite. However, after the FRCM jacket was removed, it was observed that diagonal cracks formed in the beam beneath the composite (see Figure 4). In Figure 4, it is also observed that above the diagonal crack, the fibers and the external matrix layer debonded from the internal matrix layer. However, below the diagonal crack, the entire FRCM jacket detached from the concrete substrate. Beam S1-FRCM-F4-UA failed by diagonal tension and local detachment of the FRCM system between the anchors.

![Figure 4 Cracking pattern for beam S1-FRCM-F4-UN](image)

**COMPARISON BETWEEN EXPERIMENTAL AND ANALYTICAL RESULTS**

The model proposed by Triantafillou and Papanicolaou (2006) to predict the additional shear strength provided by the FRCM system ($V_{f\text{-pred}}$, see Eq. 1) was first presented for fully wrapped rectangular beams without anchors and then extended for U-wrapped elements (Tzoura and Triantafillou, 2015). The effective stress in the FRCM system ($\sigma_{\text{eff}}$) is computed based on the average strain reached across the shear crack, taken as 50% of the ultimate strain of the bare fibers, $\varepsilon_{fu}$:

$$
V_{f\text{-pred}} = \rho_f \sigma_{\text{eff}} b_w d_f; \quad \sigma_{\text{eff}} = 0.5E_f \varepsilon_{fu}
$$

where $d_f$ is the effective depth of the jacket, taken as 0.9d, and $\rho_f$ is FRCM reinforcement ratio ($2t_f/b_w$, $t_f$= nominal fiber thickness). In Table 4, the values of $V_{f\text{-pred}}$ computed with Eq. 1 for the strengthened beams without anchors are presented along with the variables used for their calculation:

<table>
<thead>
<tr>
<th>Beam</th>
<th>$b_w$ (mm)</th>
<th>$d$ (mm)</th>
<th>$d_f$ (mm)</th>
<th>$t_r$ (mm)</th>
<th>$\rho_f$</th>
<th>$\sigma_{\text{eff}}$ (MPa)</th>
<th>$V_{f\text{-pred}}$ (kN)</th>
<th>$V_d/V_{f\text{-pred}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-FRCM-F3-UN</td>
<td>150</td>
<td>250</td>
<td>225</td>
<td>0.047</td>
<td>0.0006</td>
<td>2350</td>
<td>49.7</td>
<td>0.55</td>
</tr>
<tr>
<td>S1-FRCM-F4-UN</td>
<td>150</td>
<td>250</td>
<td>225</td>
<td>0.27</td>
<td>0.0036</td>
<td>1200</td>
<td>145.8</td>
<td>0.24</td>
</tr>
</tbody>
</table>

It can be seen that the model overestimates the additional shear strength provided by the system, which indicates that the level of strain/stress predicted by the model is overestimated. The overestimation is higher for the beam strengthened with steel fibers. At this point, it is important to point out that the model was developed for FRCM composite with carbon fibers, and therefore, its applicability for other fibers needs to be studied and validated.
CONCLUSIONS

- Using FRP composites was able to change the shear failure of the control beam into a flexural failure for the strengthened beam, providing an increase in the beam capacity of 47%. There was also an increase in the displacement at mid-span corresponding to maximum load.
- Beams strengthened with carbon-FRCM composites showed an increase in shear strength of 24% and 26% for specimens without and with anchors, respectively. For beams strengthened with steel-FRCM composites, the increase was 30% irrespective of the use of anchors.
- The use of anchors did not result in a significant increase in the shear strength for either type of FRCM composites. However, strengthened beams with anchors showed larger displacements at mid-span.
- Unlike with FRP composites, cracking was visible on the surface of the FRCM jackets. However, for the case of steel-FRCM composites, after the removal of the jacket, it was observed that the cracking pattern on the surface of the jacket was different from that observed in the concrete beneath it.
- For FRCM strengthened beams without anchors, the type of failure was different depending on the type of fiber used. For carbon fibers, local detachment of the entire thickness of the jacket from the concrete substrate was observed. For steel fibers, detachment of the fibers and external matrix layer or of the entire composite from the substrate was witnessed.
- The model by Triantafillou and Papanicolaou (2006) overestimated the additional shear strength provided by the FRCM system for the beams presented in this paper. A larger overestimation was observed for steel FRCM than carbon FRCM, which might be related to the different debonding failure modes observed.

ACKNOWLEDGMENTS

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REFERENCES

A NEW SHEAR STRENGTH MODEL FOR RC DEEP BEAMS STRENGTHENED WITH FRP COMPOSITES

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ABSTRACT

Many existing reinforced concrete (RC) deep beams have been found to be shear-deficient due to insufficient shear reinforcement, substantial increment in service load, and severe environmental conditions, and thus need to be repaired. And externally bonded fiber reinforced polymer (FRP) provides an excellent solution. However, up to date, limited studies have been carried out to investigate the shear behavior of deep beams strengthened in shear with FRP composites. This paper presents a new analytical model based on strut-and-tie method for the shear strength prediction of FRP-strengthened RC deep beams. In the proposed model, the Kupfer-Gerstle biaxial failure envelope is adopted to predict shear failure of the shear span region under a stress state of biaxial tension and compression by either the compressive crushing of the strut or the diagonal tensile cracking of concrete. And the contribution of the FRP composite and the shear reinforcement in the deep beam are taken into account by increasing the nominal tensile strength of the concrete. A test database is also assembled to verify the proposed model, which demonstrates its simplicity and effectiveness.

KEYWORDS

Fiber-reinforced polymer (FRP), RC deep beams, strengthening, shear strength, biaxial compression-tension failure envelope.

INTRODUCTION

Deep beams are those members whose clear spans do not exceed four times the overall member depth, or regions of beams with concentrated loads within twice the member depth from the support (ACI-318 2008). There are several applications of RC deep beams, such as transfer girders in tall buildings, wall of water tanks, offshore structures and foundations (Zhang et al. 2004). Many existing RC deep beams have been found to be shear-deficient due to insufficient shear reinforcement, substantial increment in service load, and severe environmental conditions, and thus need to be repaired.

Externally bonded fiber reinforced polymer (FRP) composites provides an excellent solution for such an issue, due to their high strength-to-weight ratio, excellent corrosion resistance, and ease for installation. Up to date, however, most research on FRP shear strengthening has been focused on regular beams (Lee et al. 2011), and limited studies have been carried out on FRP-strengthened deep beams in shear (Zhang et al. 2004; Islam et a. 2005; Lee et al. 2011), especially for the theoretical research based on rational models. A series of experimental tests carried out by Zhang et al. (2004) to demonstrate the feasibility of using an externally bonded CFRP system for increasing the shear strength of deep beams and a corresponding design method was also developed. Islam et al. (2005) also carried out a series of experimental tests to improve shear behaviour of deficient deep beams using an externally bonded FRP system and found that the shear strength of the deep beam was enhanced. Recently, Lee et al. (2011) also investigate the performance of shear-deficient RC T-section deep beams strengthened with CFRP sheets through tests, and concluded from the test results that the key variables of strengthening length, fiber direction combination, and anchorage have significant influence on the shear performance of strengthened deep beams. Code design methods were also adopted to evaluate the shear strengths.

This paper presents a new analytical model based on strut-and-tie model (STM) for the shear strength prediction of FRP-strengthened RC deep beams. In the proposed model, the Kupfer-Gerstle biaxial failure envelope (Kupfer and Gerstle, 1973) is adopted to predict shear failure of the shear span region under a stress state of biaxial tension and compression by either the compressive crushing of the strut or the diagonal tensile cracking of concrete. And the contribution of the FRP composite and the shear reinforcement in the deep beam are taken into account by
increasing the nominal tensile strength of the concrete. A test database is also assembled to verify the proposed model.

**STRUT-AND-TIE MODEL FOR RC DEEP BEAMS**

A STM for simply supported RC deep beams under two-point symmetrical top loading is given in Fig. 1, modified from STM for pre-tensioned concrete (PC) deep beams (Wang and Meng 2008), which consists of the upper nodal zone A, the bottom nodal zone B, the strut AB, the concrete horizontal strut and the tension tie. The failure of the strut AB is identified as the ultimate limit state of the model. In the model, \( h_0 \) is the effective depth of the beam; \( a \) is the shear span measured between the concentrated load and the support centre; \( e \) is the distance from the support centre to the beam end (Fig. 1).

**Basic Assumptions and Failure Criterion**

The bottom nodal zone B suffers the greatest compressive and tensile stresses simultaneously, and thus it would determine the shear strength of the model. Therefore, in the proposed model, it is also assumed that the shear strength of the FRP-strengthened RC deep beams is reached when the stresses of concrete at the nodal zone B reach the failure envelope, as previously adopted for PC deep beams (Wang and Meng 2008). Besides, the reinforced concrete in the deep beam is idealized as an equivalent homogenous material in a biaxial plane stress state and the contribution of the FRP and web steel reinforcement is accounted for by increasing the nominal tensile strength of the idealized homogenous material. This approach has been used to predict the shear strength of PC deep beams and RC beam-column joints (Wang et al. 2012).

It is well known that for concrete in a biaxial stress state, the appearance and widening of tensile cracks in concrete lead to deteriorations of its compressive strength along the crack direction. To account for the softening effect, Kupfer and Gerstle (1973) proposed the Kupfer-Gerstle’s biaxial compression-tension failure criterion (Fig. 2), which seems either more convenient than the principal tensile strain methods, or more theoretical than the efficiency factor methods. Considering the fact that the tensile stress is great enough at shear failure, the small curved segment, AC, (Fig. 2) of the failure criterion is ignored, while the BC segment (Fig. 2) is utilized to account for the effect of tension on the compressive strength of concrete in the nodal zone B in the proposed model, just expressed as below.

\[
f_{t,i} - 0.8 \frac{f_{c}}{f_{c}^{'}} = 1
\]

(1)

where \( f_i \) and \( f_c \) are respective the principal tensile and compressive stresses, acting perpendicular and along to the axis of the diagonal strut AB, respectively, and the tensile stress is taken as positive; \( f_c^{'} \) is the concrete compressive strength of cylinder, representing the maximum compressive strength in the \( f_i \) direction; \( f_{t,i} \) is the nominal tensile strength of concrete considering both contributions from the steel shear reinforcements and the FRP composite in the \( f_i \) direction (Fig. 3). Detailed derivation information for each term will be given in the following text.
Determination of Terms $f_1$, $f_2$ and $f_{t,n}$

From the equilibrium of forces at the nodal zone $B$ (Fig. 1),

$$F_c = V_n \sin \theta$$  \hspace{1cm} (2)

$$T = F_c \cos \theta = V_n \tan \theta$$  \hspace{1cm} (3)

where $F_c$ and $T$ are the forces in the strut and bottom tension tie, respectively; $\theta$ is the inclined angle of the strut (Fig. 1).

Thus, from Eq. (2), the principal compressive stress, $f_2$, in the direction of the strut at the bottom nodal zone $B$ (Fig. 3) can be computed by

$$f_2 = \frac{F_c}{A_{str}}$$  \hspace{1cm} (4)

where $A_{str}$ is the cross-sectional area of the strut at the nodal zone $B$ and is defined as

$$A_{str} = b_w (l_a \cos \theta_s + l_b \sin \theta_s)$$  \hspace{1cm} (5)

where $b_w$ is the width of the web; $l_a$ is the depth of the bottom nodal zone $B$; $l_b$ is the width of the support-bearing plates (Fig. 1).

As for the principal tensile stress, $f_1$, at the bottom nodal zone $B$ (Fig. 3), it was calculated as follows, as adopted for PC deep beams (Wang et al. 2008)

$$f_1 = \frac{kT \sin \theta_s}{A_c / \sin \theta_s} = kp$$  \hspace{1cm} (6)

where $p$ is the average equivalent tensile stress across the strut and $A_c$ is the cross-sectional area of beam. $k = 4$ for the case of bottom longitudinal reinforcement which induces a linear stress distribution along the strut (Wang et al. 2008). As for web steel reinforcement and FRP composites, which is commonly uniformly distributed, it is noteworthy that the rectangular stress block will be induced along the strut, that is, $k = 1$ can be obtained without any stress distribution assumption.

In a similar fashion as Eq. (6), the combined tensile capacity, $f_{t,n}$, at the bottom nodal zone $B$ (Fig. 3) can be expressed as below, in which the first, second, third and fourth terms are respectively the contributions of the
bottom longitudinal reinforcement, \( f_{ls} \), web steel reinforcements (horizontal, vertical or inclined), \( f_{ws} \), the FRP, \( f_{frp} \), and the concrete, \( f_{cu} \),

\[
f_{tu} = f_{ls} + f_{ws} + f_{frp} + f_{tc}
\]

\[
f_{tu} = \frac{kA_{j}f_{j} \sin \theta_{j}}{\sum_{i} A_{i} / \sin \theta_{i}} + \frac{A_{w}f_{yw} \sin (\theta_{w} + \theta_{n})}{A_{w} / \sin \theta_{i}} + \sum_{i} A_{j}E_{ij} \epsilon_{le,i} \sin (\theta_{j} + \theta_{i}) + f_{ct}
\]

where \( A_{i} \) and \( A_{w} \) are the respective total areas of longitudinal and web reinforcement crossing the strut; \( f_{j} \) and \( f_{yw} \) are the respective yield strengths of longitudinal and web reinforcement; \( \theta_{e} \) is the angle of the direction of the resultant force of all web reinforcement (vertical, horizontal or inclined) crossing the strut to the horizontal axis (Fig. 1); the contribution of concrete to the nominal tensile strength, \( f_{cu} \), is given by ACI318 (2008):

\[
f_{cu} = 0.556 \sqrt{f_{c}^{0}}
\]

In Eq. (7), \( A_{ij} \) and \( E_{ij} \) is the cross-sectional area and elastic modulus of the FRP strips crossing the strut at an angle \( \theta_{ij} \) with respect to the horizontal axis, respectively. The FRP contribution, \( f_{frp} \), can be estimated as the sum of the contributions of the strips in different directions. As shown in Fig. 3, the total tension force in the generic direction can be expressed by

\[
F_{ij} = A_{ij}E_{ij} \epsilon_{le,ij}
\]

where \( A_{ij} \) can be expressed by Eq.(10),

\[
A_{ij} = 2 \frac{h_{le,i} (\cot \theta_{e} + \cot \theta_{ij})}{s_{ij}} w_{ij} F_{ij}
\]

Where \( h_{le,i} \) is effective height of the strip bonded on the web at ultimate; \( w_{ij} \) is the strip width; \( s_{ij} \) is the distance between two strips; \( t_{ij} \) is the equivalent thickness of strips (Fig. 4).

In Eq. (7), \( \epsilon_{le,ij} \) is the effective FRP strain at ultimate and can be computed by Eq. (11), which was originally developed for intermediate crack debonding in FRP-strengthened RC beams (Teng et al. 2004).

\[
\epsilon_{le,ij} = 0.36 \tau_{max,i} / \sqrt{t_{ij} E_{ij}}
\]

\[
\tau_{max,i} = 1.5 \beta_{w,i} f_{cu}
\]

\[
\beta_{w,i} = \frac{2.25 - w_{ij} / (s_{ij} \sin \phi_{ij})}{1.25 + w_{ij} / (s_{ij} \sin \phi_{ij})}
\]

In the case of continuous FRP composite (i.e. sheet), it can be taken to be composed of strips with no space between them, and thus the above same derivation can be applicable just with \( s_{ij} \) equal to \( w_{ij} \).

In Eq. (7), it is assumed that the web steel reinforcements have reached their respective yield stresses in calculating the nominal tensile strength of concrete. The simple summation of the four components as proposed herein leads to only slight underestimation with an acceptable overall accuracy as shown later in the paper.

**Shear Strength of FRP-Strengthened RC Deep Beams**

From Eqs. (1) ~ (11), the following expression can be derived for \( V_s \):

\[
V_s = \frac{1}{k \sin 2\theta_{k} + 0.8 \frac{2f_{tu} A_{k}}{f_{c} A_{w} \sin \theta_{i}}}
\]
Besides, from the equilibrium of forces at the nodal zone A (Fig. 1),

\[ b_n f_c' = F_c \cos \theta_s \] (13)

Substituting for \( F_c \) from Eq. (2), the depth of the nodal zone A, \( l_c \), can be derived:

\[ l_c = \frac{V_n}{b_n f_c' \tan \theta_s} \] (14)

\( \theta_s \) can be computed from

\[ \tan \theta_s = \frac{h - \frac{l_a}{2} - \frac{l_c}{2}}{a} \] (15)

From Eqs. (14) and (15), the term \( l_c \) cannot be determined initially, so an iterative procedure is required. For convenience and simplicity, the interactive procedure can be simplified by assuming \( l_c = l_a \), and the error introduced in \( V_n \) is generally less than 2%, which is in accordance with the findings of previous researchers [Wang and Meng 2008].

From above all, the steps for determining the nominal shear strength, \( V_n \), can be summarized as follows:

Step 1. Determine \( \theta_s \) from Eq. (15), assuming \( l_c = l_a \), where \( l_a = 2(h - h_0) \).

Step 2. Determine \( f_{ts} \) from Eq. (7).

Step 3. Calculate \( V_n \) from Eqs. (12).

**VERIFICATION OF THE PROPOSED SHEAR STRENGTH MODEL**

The proposed model was validated using 20 simply supported FRP-strengthened RC deep beams from the literature (Zhang et al. 2004; Islam et al. 2005). All specimens have a rectangular cross-section, and eventually failed in shear with or without FRP debonding. They had a compressive cylinder strength varying from 38MPa to 43 MPa, an overall depth between 229 mm and 800 mm, and a shear span to depth ratio, \( a/h_0 \), from 0.81 to 1.88. The vertical and horizontal web reinforcement ratios ranged from zero to 0.2%, and zero to 0.2%, respectively. Different CFRP systems have been used to strengthen the deep beams, such as fibre wrap, strips and grids.

The test shear force, \( V_{n, test} \), and the model prediction, \( V_{n, model} \), are shown in Fig. 5 for the proposed model. The test-to-predicted average strength ratio (AVG) and its coefficient of variation (COV) are 1.08 and 0.119, respectively, which indicates that the prediction is not only accurate, consistent, but also safe. And it further indicated that the assumptions adopted for them are sufficiently accurate for the model predictions.

![Figure 5 V_n, test vs. V_n, model for proposed model](image-url)
CONCLUSIONS

This paper has proposed a modified strut-and-tie model for predicting shear strength of FRP-strengthened RC deep beams. A simple and reasonable formula has also been derived based on the Kupfer-Gerstle biaxial failure envelope. The contribution of the external bonded FRP composites is taken into account by increasing the nominal tensile strength of the concrete. And the effective FRP strain is accounted using the approach originally developed for the intermediate crack debonding in FRP-strengthened RC beams. The predictions of the proposed model were evaluated with of 20 test results, which demonstrated that the predictions are accurate, consistent and conservative and further indicated that the assumptions adopted for them are sufficiently accurate for the model predictions.

ACKNOWLEDGEMENTS

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SHEAR STRENGTHENING OF CONTINUOUS REINFORCED CONCRETE
T-BEAMS USING DEEP EMBEDMENT TECHNIQUE

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ABSTRACT

Despite numerous studies, shear behaviour before or after strengthening is still not fully understood, particularly in continuous concrete structures which are the norm. Upgrading shear resistance is altogether more difficult since Externally Bonded Reinforcement (EBR) or Near Surface Mounted (NSM) techniques do not allow the FRP material to be anchored into the compression zone of the T-beams and they cannot be used in cases where the sides of the beams are inaccessible. An innovative retrofit technique, named Deep Embedment (DE) or Embedded Through Section (ETS) technique involves the insertion of FRP/steel bars upwards into vertical or inclined holes which have been drilled from the soffit of concrete beams. In this way, the tension and compression chords of the beams are directly connected while the bars are bonded to the concrete core through adhesives. With this technique strengthening can be done in cases where the webs are inaccessible. Thus the main focus of this study is to significantly contribute to the current knowledge on the behaviour of Reinforced Concrete continuous T-beams strengthened in shear using the DE technique where large shear forces are combined with large negative bending moments. An experimental program consisting of ten two-span continuous T-beams designed to fail in shear was carried out in order to significantly contribute to the current knowledge on the behaviour of RC continuous T-beams strengthened in shear using this technique. Therefore, this paper reports on the test results and on their significance in being able to apply this technique on concrete structures by validating them through adequate analytical models.

KEYWORDS

Shear resistance, retrofitting, fiber-reinforced polymers, reinforce concrete T- beams, deep embedment.

INTRODUCTION

Strengthening and rehabilitation of reinforced concrete (RC) structures has become an important part of civil engineering. Gold and Martin (1999) emphasised the significant redundant building space in the UK to be strengthened, much of which was constructed in the 1960s and 1970s. Majority of the buildings from 1980s, 1990s and pre-and-post World War II need to be adapted to meet the requirements of the 21st century. Shortage of structural ductility can be the cause of the brittle and catastrophic failure of a structure. To date recent codes, require a high amount of shear reinforcement which differs from previous codes that did not obtain strict rules for the increased ductility (spacing and lack of concrete cover, irregular stirrups). Triantafillou 1998; Khalifa et al. 1998; Pellegrino and Modena 2002; Chaallal et al. 2011, have shown shear capacity of RC beams can be considerably enhanced by using the Externally Bonded technique (EB). (De Lorenzis and Nanni 2001; Barros and Dias 2005; Chaallal et al. 2011, have presented results in Near Surface Mounted (NSM) technique. These techniques have been developed due to the dangerous and sudden shear failure within reinforced concrete (RC) structures. Lack of understanding concerning the shear behaviour before and after strengthening still exists within research, even though use of FRP reinforcement on the basis of experimental research on simply supported beams is defined by the International and national guidelines and codes (Concrete Society Technical Report 55, ACI, etc.). This is prominent for continuous concrete structures, which have been disregarded in the previous studies. Improvement of shear resistance is altogether more challenging by using Near Surface Mounted (NSM) and Externally Bonded Reinforcement (EBR) techniques which are broadly used for flexural strengthening. It is not realistic choice to full FRP wrapping of beams as they are frequently cast monolithically with the top slab. So, U-wrapping or side-wrapping is sometimes used. It is important to acknowledge truss action may not be mobilised.
due to these methods as FRP material cannot be anchored into compression zones; similarly, the impact of debonding of FRP laminates within low strains raises additional problems. In scenarios were sides of beams are unreachable the external bonding techniques cannot be used. The behaviour within FRP shear reinforcement in the negative moment region in continuous structures has not yet been fully understood; more research has been conducted on simply supported beams employing FRP shear strengthening in the positive moment regions. Large shear forces co-exist with large negative bending moments at the same location; as shown in Figure 1.

An alternative strategy for shear strengthening of reinforced concrete beams with FRP/steel bars, Deep Embedment (DE) or Embedded Through Section (ETS) has been developed (Valerio and Ibell, 2003; Valerio et al. 2009; Chaallal et al. 2011; Mofidi et al. 2012). In this technique, DE bars are epoxy bonded into previously drilled holes (vertical or inclined) through the cross section of the RC beams. In this way, tension and compression chords are directly connected and bond between FRP bars and concrete is much better, which makes this technique superior in comparison with Externally Bonded (EB) and Near Surface Mounted (NSM) methods (Valerio et al. 2009), (Figure 2). The novelty of this research is focused on how the deep embedment technique can be extended in scope by considering strengthening materials other than CFRP (GFRP and steel are being considered), angles of drilling other than vertical (angles at 45 degrees are being considered), and the effects of continuity on shear strengthening. An experimental program consisting of ten RC continuous beams has been conducted in order to provide useful data for a deeper understanding of failure mechanisms in DE strengthened continuous beams as well as the contribution of FRP/steel bars to its total shear capacity. This experimental program will be followed by the development of an analytical model for the calculation of the shear contribution of DE bars. The experimental program will be described in detail and obtained results presented.

**TEST PROGRAM**

The experimental program involves tests performed on ten full-scale reinforced concrete continuous T-beams. They were all designed according to the British Standards (BS 8110-1). Average geometry of typical continuous reinforced concrete beams in buildings was considered while adopting dimensions of the specimens. All test beams have the same dimensions and internal reinforcement as shown in Figure 3 and Figure 4. Their characteristics are summarised in the Table 1. Reinforcement ratios in hogging and sagging zones were kept constant at 1.31% in all beams. Stirrups were spaced at a distance, s=d/2 which corresponds to a shear reinforcement ratio $\rho_{sw} = 0.1\%$.

<table>
<thead>
<tr>
<th>Load</th>
</tr>
</thead>
</table>

![Figure 3 Longitudinal cross section](image)
Table 1 Dimensions of the beams

<table>
<thead>
<tr>
<th></th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam length L [mm]</td>
<td>3840</td>
</tr>
<tr>
<td>Span 1 L₁ [mm]</td>
<td>1290</td>
</tr>
<tr>
<td>Span 2 L₂ [mm]</td>
<td>2400</td>
</tr>
<tr>
<td>Flange width bᵢ [mm]</td>
<td>350</td>
</tr>
<tr>
<td>Beam height h [mm]</td>
<td>350</td>
</tr>
<tr>
<td>Web thickness bₚ [mm]</td>
<td>150</td>
</tr>
<tr>
<td>Flange thickness hᵢ [mm]</td>
<td>100</td>
</tr>
<tr>
<td>Effective depth d [mm]</td>
<td>320</td>
</tr>
<tr>
<td>Shear span-to-effective depth ratio a/d [mm]</td>
<td>3</td>
</tr>
</tbody>
</table>

Materials

The concrete grade was standard C40 grade with water-to-cement ratio of 0.53 and coarse aggregate less than or equal to 20 mm. The average values from the standard cube and tensile splitting tests have shown \( f_{cu} = 60 \) MPa for the concrete compressive strength and \( f_{ct} = 4.86 \) MPa for the concrete tensile strength. Spirally wound sand-coated FRP bars and steel bars have been used for specimen strengthening together with two-component adhesive. Their properties are given in Table 2.

Table 2 Material properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Tensile strength ( f_{fu} ) [MPa]</th>
<th>Modulus of elasticity ( E_f ) [GPa]</th>
<th>Ultimate strain [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aslan 200 CFRP bar</td>
<td>2172</td>
<td>124</td>
<td>1.81</td>
</tr>
<tr>
<td>Aslan 100 GFRP bar</td>
<td>827</td>
<td>46</td>
<td>1.94</td>
</tr>
<tr>
<td>Steel bar</td>
<td>500</td>
<td>210</td>
<td>/</td>
</tr>
<tr>
<td>Hilti HIT-RE500 Epoxy resin</td>
<td>43.5</td>
<td>1.49</td>
<td>2.00</td>
</tr>
</tbody>
</table>

Strengthening of the beams

As illustrated in the Table 3 and Figure 5, all the beams are divided into three groups (I, II and III). The control specimen, not strengthened with Deep Embedded bars, is labelled as CON. Each group consists of three beams strengthened in the shear zone using three different configurations: a) vertical bars spaced at 150 mm (C150, G150 and S150 - one bar between two shear links), b) vertical bars spaced at 75 mm (C75, G75 and S75 - two bars between two shear links) and c) inclined bars (45°) at 150 mm (C150∠, G150∠ and S150∠ - each bar crossing two shear links).

Table 3 Description of test specimens

<table>
<thead>
<tr>
<th>Group</th>
<th>Number of beams</th>
<th>Bar type</th>
<th>Bar diameter ([\text{mm}])</th>
<th>Mark</th>
<th>Bar spacing ([\text{mm}])</th>
<th>Angle ([\degree])</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>1</td>
<td>/</td>
<td>/</td>
<td>CON</td>
<td>/</td>
<td>/</td>
</tr>
<tr>
<td>I</td>
<td>3</td>
<td>CFRP</td>
<td>6</td>
<td>C150</td>
<td>150</td>
<td>90</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>C75</td>
<td>75</td>
<td>90</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>C150∠</td>
<td>150</td>
<td>45</td>
</tr>
<tr>
<td>II</td>
<td>3</td>
<td>GFRP</td>
<td>6</td>
<td>G150</td>
<td>150</td>
<td>90</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>G75</td>
<td>75</td>
<td>90</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>G150∠</td>
<td>150</td>
<td>45</td>
</tr>
<tr>
<td>III</td>
<td>3</td>
<td>STEEL</td>
<td>6</td>
<td>S150</td>
<td>150</td>
<td>90</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>S75</td>
<td>75</td>
<td>90</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>S150∠</td>
<td>150</td>
<td>45</td>
</tr>
</tbody>
</table>
PRESENTATION AND DISCUSSION OF EXPERIMENTAL RESULTS

Table 4 summarizes the experimental results obtained from the tests for all the test groups. The results are presented in terms of the loads attained at failure, experimental shear resistance, deflection at load point and the type of the failure. A simplified methodology accepted in the TR55 (Design guidance for strengthening concrete structures using fibre composite materials) was adopted in order to calculate the contributions of Deep Embedded bars to the shear resistance of the beams. ACI318 (Building Code Requirements for Structural Concrete and Commentary) and BD44 (The assessment of concrete highway bridges and structures) gave the closest predictions for the shear capacities of the beams.

Table 4 Description of test specimens

<table>
<thead>
<tr>
<th>Group</th>
<th>Specimen</th>
<th>Load at failure [kN]</th>
<th>Shear resistance [kN]</th>
<th>Shear capacity increase [%]</th>
<th>Deflection at load point [mm]</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>CON</td>
<td>184</td>
<td>135</td>
<td>0</td>
<td>7.6</td>
<td>Shear</td>
</tr>
<tr>
<td>I</td>
<td>C150</td>
<td>271</td>
<td>181</td>
<td>34</td>
<td>15.7</td>
<td>Shear</td>
</tr>
<tr>
<td>I</td>
<td>C75</td>
<td>315</td>
<td>219</td>
<td>62</td>
<td>20</td>
<td>Flexure</td>
</tr>
<tr>
<td>I</td>
<td>C150∠</td>
<td>320</td>
<td>222</td>
<td>64</td>
<td>19</td>
<td>Shear</td>
</tr>
<tr>
<td>II</td>
<td>G150</td>
<td>304</td>
<td>205</td>
<td>52</td>
<td>20</td>
<td>Shear</td>
</tr>
<tr>
<td>II</td>
<td>G75</td>
<td>322</td>
<td>222</td>
<td>64</td>
<td>25</td>
<td>Flexure</td>
</tr>
<tr>
<td>II</td>
<td>G150∠</td>
<td>280</td>
<td>191</td>
<td>41</td>
<td>14</td>
<td>Shear</td>
</tr>
<tr>
<td>III</td>
<td>S150</td>
<td>269</td>
<td>183</td>
<td>35</td>
<td>12</td>
<td>Shear on the right side of the load</td>
</tr>
<tr>
<td>III</td>
<td>S75</td>
<td>273</td>
<td>188</td>
<td>39</td>
<td>9</td>
<td>Shear</td>
</tr>
<tr>
<td>III</td>
<td>S150∠</td>
<td>231</td>
<td>160</td>
<td>18</td>
<td>7</td>
<td>Flexure</td>
</tr>
</tbody>
</table>

Table 4 shows that the shear-strengthened beams experienced significant increase in capacity with respect to the control beam. In this experimental program, the average increase in shear capacity reached around 55%. Deep Embedment strengthening system can significantly enhance the shear capacity of RC beams even in presence of a minimum amount of transverse steel reinforcement. Decreasing the DE bar spacing resulted in a higher contribution to shear resistance (specimens C75 and G75 failed in flexure). Specimens strengthened with inclined DE bars reached similar ultimate capacities as those strengthened with DE bars spaced at 75mm.

Figure 6 Shear cracks in specimen CON

Figure 7 Shear cracks in specimen C150∠

Figure 8 Shear cracks in specimen G150∠

Figure 9 Shear cracks in specimen S75
Specimens CON, C150, C150\(\angle\), G150, G150\(\angle\) and S75 failed in shear whereas specimens C75 and G75 failed in flexure. The load at which the first diagonal cracks appeared was of a similar level for all beams. Diagonal cracks first started to develop on the left side of the load and then followed by parallel crack formation on the right side of the beam middle support (pin). Characteristic shear crack patterns are shown in the Figures 6, 7, 8 and 9.

Figures 10, 11 and 12 show the curves for applied load versus maximum displacement at the point load for the control and strengthened beams. They represent typical behaviour during a shear test. A loss of a beam’s stiffness caused a redistribution of internal stresses and activated shear links and DE bars to increase shear resistance.

![Figure 10 Load versus displacement – group I](image1)

![Figure 11 Load versus displacement – group II](image2)

![Figure 12 Load versus displacement – group III](image3)

DE technique showed to be very efficient in developing high strains in DE bars before the final failure happens, some of the DE bars experienced very high strains during the tests. Characteristic are samples G150, G150\(\angle\) and C150\(\angle\) where DE bars almost reached their ultimate tensile capacities. Following figures represent some of the damaged bars (Figures 13, 14 and 15).

![Figure 13 GFRP DE bar in specimen G150\(\angle\) (1.6% strain)](image4)

![Figure 14 CFRP DE bar in specimen C150\(\angle\) (1.5% strain)](image5)

![Figure 15 Steel DE bar in specimen S75](image6)

**CONCLUSIONS**

This paper presents the results of an experimental investigation involving nine tests on RC T-beams strengthened in shear using the Deep Embedment technique. On the basis of the results of the present research, the following main conclusions can be drawn:

- DE technique has been demonstrated to be effective and reliable for enhancing the shear capacity of continuous concrete beams.
The DE technique is very promising; therefore, further data analysis is needed to address other important effects along with the development of suitable analytical models for safe use of this retrofit technique.

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REFERENCES


SIZE EFFECT IN RC BEAMS SHEAR-STRENGTHENED WITH FRP

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ABSTRACT

Existing research has shown that externally bonded fibre-reinforced polymer (FRP) reinforcement is an effective shear strengthening technique for reinforced concrete (RC) beams. One issue that has not been properly clarified by existing research is the size effect in RC beams shear-strengthened with FRP. Most of the existing experimental studies have been carried out on beams of small/medium size (e.g., with a beam height < 0.45m) and most of the existing shear strength models have been developed and substantiated using the test results of such small/medium-size beams. This paper presents the test results of 18 beams in 3 series corresponding to three different beam heights respectively: 300 mm, 600mm and 900 mm. Each series consisted of 6 beams, including a group of 3 beams with steel stirrups and another group of 3 beams without steel stirrups. Each group had 3 beams: an RC beam without strengthening (control specimen), a beam shear-strengthened with FRP U-strips, and a beam shear-strengthened with FRP full wraps. All the 18 beams had a shear span-to-depth ratio of 3.0. The test results showed that a significant size effect exists in RC beams shear-strengthened with FRP U-strips while the size effect is minimal when FRP full wraps are used. The test results also demonstrated that significant shear interactions exist among the different components (concrete, steel and FRP) of an FRP-strengthened RC beam.

KEYWORDS

FRP, RC beams, strengthening, shear strength, size effect, shear interaction, U-strips, full wraps.

INTRODUCTION

Extensive research has shown that externally bonded (EB) fibre-reinforced polymer (FRP) reinforcement provides an effective method for the shear strengthening of a reinforced concrete (RC) beam (Teng et al. 2002). It is also well-known that concrete is a quasi-brittle material in which localized cracking/damage associated with material softening is a common phenomenon; as a result, the size effect exists in the shear strength of RC beams: the nominal shear strength \(\tau\), defined as the ultimate shear force in the beam divided by its effective sectional area (= beam width \(b\) x effective beam height \(h_0\)), decreases with an increase in beam height (Bazant 1999). Steel reinforcements (e.g. stirrups) can substantially mitigate but cannot eliminate the size effect (Yu and Bazant 2011). In an RC beam shear-strengthened with an EB FRP system, the FRP-to-concrete bonded interface is the weakest link and its behaviour has a major effect on the shear strength of the strengthened beam. It is well known that for an FRP-to-concrete bonded interface, an effective bond length exists beyond which the force transfer by the interface does not increase (Chen and Teng 2001). Because of the existence of an effective bond length, the force transfer between concrete and EB FRP is limited to a localized zone, which is initially within a mobilized zone near the most stressed part of the FRP and then propagates towards its less stressed part (Yuan et al. 2004). This force transfer mechanism between FRP and concrete is another source of size effect in RC beams shear-strengthened with EB FRP. As a result, the size effect in RC beams shear-strengthened with EB FRP is complex and of great importance to structural safety.

Despite the obvious importance of size effect in RC beams shear-strengthened with EB FRP as explained above, most of the existing experimental studies on the subject have been conducted on small-size beams (i.e. with a
beam highest not exceeding 450 mm), and most of the existing shear strength models have been developed on the basis of small-size beam tests (Bousselham and Challall 2013). Very few studies have been carried out on size effect in FRP shear-strengthened RC beams. Although these studies have clearly shown the existence of a significant size effect in RC beams shear-strengthened FRP U-strips (Leung et al. 2007; Godat et al. 2007; Bae et al. 2012; Bousselham and Challall 2013; Long and Rovnak 2015) and a marginal size effect in RC beams shear-strengthened with FRP wraps (Leung et al. 2007), they suffer from two main limitations: 1) the beam sizes investigated do not cover the wide range of practical beam dimensions well, with the largest effective height investigated being 660 mm (beam height 720 mm) (Leung et al. 2007); 2) there is a lack of systematic, in-depth research into the mechanism behind this size effect.

To correct the deficiencies in existing knowledge, a large experimental programme has been undertaken by the authors’ research group over the past four years: a total of 30 beams were tested to examine the effects of the following parameters in depth: beam height (300mm, 600mm or 900mm), shear span-to-depth ratio (1.5 or 3.0), strengthening configuration (FRP U-strips or FRP full wraps), and presence of steel stirrups (with or without stirrups). Of these 30 test beams, 12 beams had a shear span-to-depth ratio of 1.5 while 18 beams had a shear span-to-depth ratio of 3.0. Due the limited space, only the main results of the latter beams are presented in the present paper, but readers can refer to Chen et al. (2016a) for the results of the former beams. An additional merit of the present experimental study is that the strains in both the FRP reinforcement and the steel stirrups were measured using a large number of strain gauges to allow an accurate, quantitative assessment of the contributions to shear resistance by the different components (FRP, concrete and steel stirrups) of the strengthened beam. These strain data provide useful insight into shear interactions among the three components, allowing the mechanism of size effect in RC beams shear-strengthened with EB FRP to be clarified.

**EXPERIMENTAL PROGRAMME**

**Test Specimens**

The main details and results of the beams with a shear span-to-depth ratio equal to 3 are listed in Table 1; further details can be found elsewhere (Chen et al. 2016b, 2016c). The 18 test beams are divided into 3 series in terms of beam heights: 300 mm, 600mm and 900 mm, corresponding to effective beam heights of 250 mm, 515 mm and 800 mm respectively. Each series included 6 beams, which can be further divided into 2 groups of beams with and without steel stirrups. Each group comprised 3 beams: an RC beam without FRP strengthening (control specimen), an RC beam shear-strengthened with FRP U-strips, and an RC beam shear-strengthened with FRP full wraps (also referred to simply as “FRP wraps” for brevity). The beams with and without steel stirrups were cast at two different times using commercial concrete available from a local concrete provider; the average concrete cylinder (150 mm x 300 mm) strengths tested at 28 days and on the days of testing the beams were 35.9 MPa and 45.3 MPa respectively for beams without stirrups and 36.9 MPa and 55.4 MPa respectively for beams with stirrups. To ensure that shear failure would be critical, all the beams were over-reinforced in flexure, leading to longitudinal tension steel bar ratios of 4.29%(2D32, i.e., two bars with a diameter of 32 mm), 4.12%(8D32) and 3.91%(14D36) for the beams with a height of 300 mm, 600mm and 900mm, respectively. To allow better observations of failure modes and crack patterns, horizontally discrete U strips or full wraps were bonded on the RC beam following a typical wet lay-up procedure at least 7 days before testing; careful surface preparation was adopted to ensure a good-quality bond between concrete and FRP (i.e., debonding of EB FRP occurred only in the concrete adjacent to the bonded interface, see Figure 3). The sectional corners of all the beams were rounded to a radius of around 25 mm to preclude premature FRP rupture there. To study the size effect, all the other geometrical parameters of the beams, including the thickness, width and spacing of the FRP strips, were proportional to the beams size.

**Test Set-up and Instrumentation**

All beam specimens were subjected to two concentrated loads (i.e., 4-point bending) and tested in the structural laboratory of Guangdong University of Technology using a 5,000 kN-capacity loading frame (Figure 1). Loading was applied using a manually operated actuator together with a hydraulic jack with a load capacity of 3,200 kN. During the loading process, the forces at the two loading points were each recorded by a load cell with a load capacity of 3,000 kN. Five LVDTs were installed at mid-span, the two loading points and the two supporting points respectively to measure displacements. A large number of strain gauges were used to measure strains in the FRP strips and the steel bars (see Figure 2 for their locations). Additional strain gauges were used to monitor concrete strains on the beam compression face and the strains in the tension steel bars. Furthermore, a crack meter was used to record the widths of main shear cracks at a number of locations. To exclude local compression failure at both the loading points and the supporting points, tailored-made steel loading pads [see Chen et al. (2016a, 2016b, 2016c) for details] were deployed at these locations. Of the two shear spans of a
beam, only one shear span was the target of the experimental study, while the other shear span was heavily strengthened to avoid failure. All discussions in the present paper are concerned with the test shear span with the non-test shear span basically ignored.

Table 1 Details and main results of beam specimens

<table>
<thead>
<tr>
<th>Beam No.</th>
<th>h (mm)</th>
<th>d (mm)</th>
<th>b (mm)</th>
<th>l0 (mm)</th>
<th>a (mm)</th>
<th>CFRP strips</th>
<th>Steel stirrups</th>
<th>( V_{\text{test}} ) (kN)</th>
<th>( V_{\text{nom}} ) (kN)</th>
<th>( \Delta V ) (%)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>SB-C</td>
<td>300</td>
<td>250</td>
<td>150</td>
<td>2.25</td>
<td>750</td>
<td>—</td>
<td>—</td>
<td>107.0</td>
<td></td>
<td></td>
<td>S</td>
</tr>
<tr>
<td>SB-UF</td>
<td>300</td>
<td>250</td>
<td>150</td>
<td>2.25</td>
<td>750</td>
<td>0.111</td>
<td>30 90</td>
<td>114.9</td>
<td>7.9</td>
<td>7.4</td>
<td>D</td>
</tr>
<tr>
<td>SB-WF</td>
<td>300</td>
<td>250</td>
<td>150</td>
<td>2.25</td>
<td>750</td>
<td>0.111</td>
<td>30 90</td>
<td>167.7</td>
<td>60.7</td>
<td>56.7</td>
<td>R</td>
</tr>
<tr>
<td>SB-S-C</td>
<td>300</td>
<td>250</td>
<td>150</td>
<td>2.25</td>
<td>750</td>
<td>—</td>
<td>—</td>
<td>R6.5@170</td>
<td>141.6</td>
<td></td>
<td>S</td>
</tr>
<tr>
<td>SB-S-UF</td>
<td>300</td>
<td>250</td>
<td>150</td>
<td>2.25</td>
<td>750</td>
<td>0.111</td>
<td>30 90</td>
<td>R6.5@170</td>
<td>188.4</td>
<td>46.9</td>
<td>33.1</td>
</tr>
<tr>
<td>SB-S-WF</td>
<td>300</td>
<td>250</td>
<td>150</td>
<td>2.25</td>
<td>750</td>
<td>0.111</td>
<td>30 90</td>
<td>R6.5@170</td>
<td>207.6</td>
<td>66.0</td>
<td>46.7</td>
</tr>
<tr>
<td>MB-C</td>
<td>600</td>
<td>515</td>
<td>300</td>
<td>3.84</td>
<td>1545</td>
<td>—</td>
<td>—</td>
<td>276.9</td>
<td></td>
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<td>S</td>
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<tr>
<td>MB-UF</td>
<td>600</td>
<td>515</td>
<td>300</td>
<td>3.84</td>
<td>1545</td>
<td>0.222</td>
<td>60 180</td>
<td>300.9</td>
<td>24.0</td>
<td>8.7</td>
<td>D</td>
</tr>
<tr>
<td>MB-WF</td>
<td>600</td>
<td>515</td>
<td>300</td>
<td>3.84</td>
<td>1545</td>
<td>0.222</td>
<td>60 180</td>
<td>622.8</td>
<td>345.9</td>
<td>124.9</td>
<td>R</td>
</tr>
<tr>
<td>MB-S-C</td>
<td>600</td>
<td>515</td>
<td>300</td>
<td>3.84</td>
<td>1545</td>
<td>—</td>
<td>—</td>
<td>R8@120</td>
<td>548.2</td>
<td></td>
<td>S</td>
</tr>
<tr>
<td>MB-S-UF</td>
<td>600</td>
<td>515</td>
<td>300</td>
<td>3.84</td>
<td>1545</td>
<td>0.222</td>
<td>60 180</td>
<td>R8@120</td>
<td>569.0</td>
<td>20.7</td>
<td>3.8</td>
</tr>
<tr>
<td>MB-S-WF</td>
<td>600</td>
<td>515</td>
<td>300</td>
<td>3.84</td>
<td>1545</td>
<td>0.222</td>
<td>60 180</td>
<td>R8@120</td>
<td>880.5</td>
<td>332.3</td>
<td>60.6</td>
</tr>
<tr>
<td>LB-C</td>
<td>900</td>
<td>800</td>
<td>450</td>
<td>5.55</td>
<td>2400</td>
<td>—</td>
<td>—</td>
<td>498.7</td>
<td></td>
<td></td>
<td>S</td>
</tr>
<tr>
<td>LB-UF</td>
<td>900</td>
<td>800</td>
<td>450</td>
<td>5.55</td>
<td>2400</td>
<td>0.333</td>
<td>90 270</td>
<td>548.6</td>
<td>49.9</td>
<td>10.0</td>
<td>D</td>
</tr>
<tr>
<td>LB-WF</td>
<td>900</td>
<td>800</td>
<td>450</td>
<td>5.55</td>
<td>2400</td>
<td>0.333</td>
<td>90 270</td>
<td>1223.3</td>
<td>724.5</td>
<td>145.3</td>
<td>R</td>
</tr>
<tr>
<td>LB-S-C</td>
<td>900</td>
<td>800</td>
<td>450</td>
<td>5.55</td>
<td>2400</td>
<td>—</td>
<td>—</td>
<td>R10@260</td>
<td>1108.3</td>
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<td>S</td>
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<tr>
<td>LB-S-UF</td>
<td>900</td>
<td>800</td>
<td>450</td>
<td>5.55</td>
<td>2400</td>
<td>0.333</td>
<td>90 270</td>
<td>R10@260</td>
<td>1202.2</td>
<td>94.0</td>
<td>8.0</td>
</tr>
<tr>
<td>LB-S-WF</td>
<td>900</td>
<td>800</td>
<td>450</td>
<td>5.55</td>
<td>2400</td>
<td>0.333</td>
<td>90 270</td>
<td>R10@260</td>
<td>1846.1</td>
<td>739.8</td>
<td>60.9</td>
</tr>
</tbody>
</table>

Note: U = U strips; W = Full wraps; F = FRP; \( h \) = beam depth (mm); \( d \) = effective depth (mm); \( b \) = beam width (mm); \( l_0 \) = net span (m); \( a \) = shear span (mm); \( s \) = stirrups; \( t_f \) = CFRP strip thickness; \( w_f \) = CFRP strip width; \( s_f \) = CFRP strip spacing; \( \Delta V \) = Percentage shear strength increase; S = Shear failure; D = FRP debonding; R = FRP rupture.

(a) Small beam (\( h = 300 \text{ mm} \))  (b) Medium-size beam (\( h = 600 \text{ mm} \))  (c) Large beam (\( h=900\text{mm} \))

Figure 1 Test set-up

TEST RESULTS AND DISCUSSIONS

Failure Modes and Shear Capacities

As detailed in Table 1, all control RC beams failed by the typical shear tension failure mode, governed by the development of a critical shear crack in the shear span. Results reported in Chen et al. (2016b, c) showed that in RC beams with steel stirrups, an apparent secondary shear crack also appeared in the shear span. The beams strengthened with FRP U-strips failed by FRP debonding (Figure 3a) while those strengthened with FRP wraps failed by FRP rupture (Figure 3b) (Table 1). It can be seen in Figure 3 that although the dominant shear cracks in beams MB-S-UF and MB-S-WF respectively are quite similar in term of crack location and angle, the crack
width of the former is much smaller than that of the latter, being 2.7 mm and 5.1 mm respectively at the ultimate state (Chen et al. 2016b, c). The smaller maximum shear crack width in RC beams strengthened with FRP U-strips implies the brittle nature of the FRP debonding failure. Furthermore, an apparent secondary shear crack appeared in the beams strengthened with FRP wraps (i.e. MB-S-WF) but not in the beam strengthened with FRP U-strips.

![Locations of strain gauges in large beams (all units in mm)](image)

(a) FRP strips  
(b) Steel stirrups

Figure 2 Locations of strain gauges in large beams (all units in mm)

Table 1 shows that the beam size affects the shear enhancement effect differently for RC beams with FRP U-strips and those with FRP wraps: for the former, the shear enhancement effect has a clear tendency to decrease with an increase in beam height, while for the latter, such a tendency does not appear to exist. It is worth noting that in medium-size beams and large beams strengthened with FRP U-strips, the shear enhancement effect became quite marginal (no more than 10%), which is in line with the results reported by Leung et al. (2007).

![Failure modes](image)

(a) FRP debonding  
(b) FRP rupture

Figure 3 Failure modes

![Nominal shear strength versus beam effective depth (size effect)](image)

Figure 4 Nominal shear strength versus beam effective depth (size effect)
Size Effect

It is a common practice to assess the size effect using the nominal strength (i.e., $v$) (Bazant 2000). The nominal shear strength of a beam is defined as the shear force at ultimate state divided by the beam effective sectional area ($= \text{beam width} \times \text{effective beam height} \times h_0$), following Bazant (2000). Figure 4 shows the nominal shear strength versus the effective beam height curves. It can be seen that the size effect is significant in RC beams without steel stirrups, which becomes less significant in RC beams with steel stirrups as expected (Yu and Bazant 2011). For both RC beams with and without steel stirrups, the FRP wraps can effectively mitigate the size effect; as a result, the size effect in RC beams strengthened with FRP wraps is insignificant or much reduced. For RC beams strengthened with FRP U-strips, however, the size effect is significant; for those RC beams without steel stirrups, the slopes of the $v$ vs. $h_0$ curves are nearly the same for the control beams and for the FRP-strengthened beams, implying that the size effect is nearly the same for these two types of beams; for RC beams with steel stirrups, the FRP U-strips appear to have substantially increased the size effect.

Shear Interactions

Using the extensive strain data obtained in the present study, the shear contributions of the different components (FRP, concrete and steel stirrups) during the loading process can be quantitatively evaluated. In the present study, the method employed by Teng et al. (2009) was adopted in calculating the shear contributions of FRP strips and steel stirrups, and only those FRP strips and steel stirrups intersected by the critical shear crack were deemed to have contributed to the shear resistance of the beam. After the shear contributions of FRP strips ($V_f$) and/or steel stirrups ($V_s$) were determined using the above method, the shear contribution of concrete can be deduced by subtracting $V_s$ (for beams without steel stirrups) or the sum of $V_f$ and $V_s$ (for beams with steel stirrups) from the total shear force. The shear interaction among the different components of FRP-strengthened RC beams can then be analysed using the results.

![Figure 5 Development of shear contributions by different components in FRP-strengthened RC beams](image)

Figure 5 shows the shear interaction process for medium-size beams without steel stirrups. It should be noted that in Figure 5, points M, N and H stand respectively for the ultimate states of control beam and FRP-strengthened beam as well as the state at which $V_f$ peaks. From Figure 5 it can be seen that important shear interactions exist between $V_f$ and $V_s$ after point M: $V_f$ increases while $V_s$ generally decreases. In RC beams strengthened with FRP wraps, this interaction is a long process that ends when the ultimate state of the strengthened beam (point N) is reached, due to the relatively large rupture strain of FRP; as a result, the total shear force increases continuously after point M, which is in sharp contrast to the immediate decrease of the shear force in the control beam. It is well established that the degradation of concrete shear resistance is associated with localized concrete cracking/damage due to material softening of concrete, which is also the source of size effect in the shear strength of RC beams. From the present test results, it can be concluded that the detrimental size effect on the shear strength can be well counteracted by the linear elastic nature of FRP as well as its large rupture strain, which explains why the FRP wraps can mitigate the size effect. In RC beams strengthened with FRP U-strips, even though such beneficial shear interactions exist in the short stage immediately after point M, $V_f$ soon starts to decrease after an initial increase due to the brittleness nature of debonding failure (i.e. debonding occurs at relatively small shear crack widths as mentioned above). In other words, the detrimental size effect associated with the material softening of concrete cannot be effectively counteracted by FRP U-strips failing by FRP debonding, which is why FRP U-strips cannot mitigate the size effect in shear-critical RC beams.
CONCLUSIONS

This paper has presented the main results of an experimental study on the size effect in RC beams shear-strengthened with either FRP U-strips or FRP full wraps. These beams all had a shear span-to-depth ratio of 3.0, which means that the behaviour of the RC beams is dominated by the formation of a major shear tension crack. From the results and discussions presented in the paper, the follow important conclusions can be obtained:

1) FRP full wraps can effectively mitigate the size effect in shear-critical RC beams, and as a result, the size effect in RC beams shear-strengthened with FRP full wraps is much reduced and can become insignificant; the relative magnitudes of shear contributions by the concrete and the FRP have a significant influence on the size effect via shear interactions;

2) FRP U strips are incapable of mitigating the size effect and may in fact significantly increase the size effect in shear-critical RC beams; as a result, the size effect of RC beams shear-strengthened with FRP U strips is significant.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the financial support provided by the National Natural Science Foundation of China (Project Nos. 51378130, 51678161) and Guangdong Natural Science Foundation (Project No. S2013010013293). The authors are also grateful for the financial support received from the Department of Education of Guangdong Province via the programme of Excellent Young College Teacher of Guangdong Province (Project No. Yq2013056) and for the financial support received by the fourth author from the National Basic Research Program of China (i.e. the 973 Program) (Project No.: 2012CB026201).

REFERENCES


A NEW CLOSED-STIRRUP TECHNOLOGY FOR SHEAR STRENGTHENING OF RC T-BEAMS USING CFRP LAMINATES ANDropes

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ABSTRACT

The objective of this study is to present a new strengthening method that prevents premature debonding of externally bonded (EB) carbon fiber-reinforced polymer (CFRP). The new method reproduces the technical advantages of full-wrap strengthening configuration, using a newly developed anchorage system. To that end, an experimental investigation on RC T-beams shear-strengthened with a new closed-stirrup technology is conducted. The closed-stirrup is made of prefabricated CFRP L-shape laminates and a CFRP rope as a closure. This application method does not use mechanical anchors and is fast and easy to install, providing a viable and cost-effective solution. It can also be useful for concrete members located in seismic regions where closed-stirrups are required for confinement purposes. The investigation involved laboratory tests performed on full-size RC T-beams. Among the features studied, the feasibility of the CFRP closed-stirrup shear strengthening technique and the influence of internal steel-stirrup ratio on the performance of EB-CFRP were examined. The test results demonstrated the effectiveness of using the closed-stirrup anchorage system in increasing the shear resistance of strengthened RC T-beams. The results also confirmed the existence of an interaction between the internal transverse steel reinforcement and the EB-CFRP composites.

KEYWORDS

RC T-beams, carbon fiber-reinforced polymer (CFRP), shear strengthening, anchorage, closed-stirrup, L-shape laminate, CFRP rope.

INTRODUCTION

The use of externally bonded (EB) carbon fiber-reinforced polymer (CFRP) to strengthen RC beams in shear is well-documented for both experimental and design aspects (e.g., Chaallal et al. 1998, Mofidi and Chaallal 2014). However, premature failure by FRP debonding due to lack of effective bond length and anchorage is still a major concern.

It is well established that shear strengthening using a full-wrap configuration offers a better behavior compared to side bonding or U-wrap schemes. However, up to date, the full-wrap technique can only be used for rectangular beams when accessible. This is obviously a serious limitation since most beams are monolithically cast with slabs and therefore have a T section. This observation has been the main impetus to conduct this experimental research study, which investigates the feasibility of shear-strengthened RC T-beams using closed-stirrups made of CFRP L-shape laminates and a CFRP rope as a closure for anchorage. Prefabricated CFRP L-shape laminates are well documented (e.g., Czaderski and Motavalli 2002, Chen and Robertson 2004, Mofidi et al. 2014). The rope is a bundle of flexible CFRP strands held together using a thin tissue net, inserted through the flange of the beam and bonded to the free ends of the L-shape laminates in a flared shape, forming a full-wrap configuration (CFRP closed-stirrup). This application method does not use mechanical anchors and is fast and easy to install, providing a viable and cost-effective solution. It considers a situation in which the CFRP is intended to increase the original shear resistance of deficient RC T-beams, to enhance anchorage and prevent premature debonding, which is a major issue particularly in shear. Moreover, such a technique will improve the confinement of concrete taking into account the whole effective depth of the beam. It can also be useful for seismic regions where closed-stirrups are required for confinement purposes.
EXPERIMENTAL INVESTIGATION

Description of specimens

A total of six tests were performed in this research study. The specimens consist of full-size RC T-beams with a total length of 4520 mm, a height of 406 mm and a shear span of 1050 mm. The details of specimens are presented in Figure 1. Table 1 presents the experimental program matrix for the test specimens. The control specimens, not strengthened with CFRP, were labeled CON, whereas the specimens retrofitted with CFRP L-shaped laminates were labeled LS-RF for the rope anchorage passing through the flange. Three different ratios of internal transverse steel were examined for each category of specimens: S0 series represents the beams with no transverse steel, whereas S3 and S1 series represent respectively the specimens with steel stirrup spaced at $s = 3d/4$ (260 mm) and $s = d/2$ (175 mm). The longitudinal tension steel reinforcement consisted of four M25 (area of 500 mm²) distributed in two layers. The compression bars consisted of six M10 (area of 100 mm²) in one layer into the flange. The transverse steel reinforcement consisted of 8 mm diameter (area of 50 mm²) for both S3 and S1 series.

Table 1 Test specimens

<table>
<thead>
<tr>
<th>Category</th>
<th>Steel-stirrup series</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>S0 (no stirrups)</td>
</tr>
<tr>
<td>Control</td>
<td>S0-CON</td>
</tr>
<tr>
<td>CFRP</td>
<td>S0-LS-RF</td>
</tr>
</tbody>
</table>

Materials

The average 28 days concrete compressive strength ($f'_c$) was 28 MPa. The longitudinal steel reinforcement has average yield strength of 470 and 540 MPa for M25 and M10 rebars, respectively. The transverse steel yield strength was 580 MPa. These values were based on tension tests performed in the laboratory according to ASTM A370-12a standard.

The CFRP closed-stirrup technique used in this study consisted of: (i) unidirectional CFRP L-shaped laminates prefabricated by pultrusion in an epoxy matrix with 0-degree orientation; and (ii) unidirectional CFRP ropes, a bundle of flexible strands held together using a thin tissue net, used as an anchorage system. The L-shaped laminates had a 90 degree bend with 25 mm inner radius, 300 longer leg x 125 mm shorter leg x 20 mm width and 2 mm thickness. They were characterized by an ultimate tensile strength of 1350 MPa, 90 GPa modulus of elasticity and 1.3% elongation at break. The CFRP ropes had a cross section area of 25.27 mm² before saturation, and were cut at 625 mm length each. They had an ultimate tensile strength of 1590 MPa, 215 GPa modulus of elasticity and 0.74% elongation at break. The CFRP L-shaped laminates were bonded to the beam surface and the ropes using a two-component adhesive, made of resin and hardener, mainly engineered for structural applications. The epoxy’s mechanical properties were: 24.8 MPa bond strength, 4.5 GPa tensile modulus of elasticity and 1% ultimate elongation.

Strengthening procedure and test setup

The CFRP closed-stirrups were distributed along the shear span in intermittent strips spaced at 175 mm (Figure 2a). The CFRP L-shape laminates were bonded transversally to the bottom and lateral faces of the web to form a U-wrap scheme, where the shorter legs of two opposite L-shape laminates were overlapped and bonded to the soffit of the beam. Afterwards, the CFRP rope was inserted through the drilled holes on both sides of the flange, at the intersection with the web, and bonded to the free ends of the L laminates as a closure to establish a full-wrap configuration, as shown in Figure 2b.
The RC T-beams were tested in three-point loading flexure. The load was applied at a distance 3d from the support to ensure a slender type beam. The tests were performed under displacement control conditions at a rate of 2 mm/min. Strain gauges were affixed to the steel-stirrups located in the loading zone along the expected plane of shear failure. The longitudinal steel rebars were also instrumented with strain gauges at the location where the load was applied. Deflections of the beams were measured using linear vertical displacement transducers installed under the load application point. The CFRP deformations were measured using strain gauges installed vertically onto each L-shaped laminate.

EXPERIMENTAL TEST RESULTS

The experimental results are summarized in Table 2. They are presented in terms of the loads reached at failure; the shear resistance due to concrete (V_c), steel stirrups (V_s), and CFRP (V_{frp}); the shear resistance gain due to CFRP; as well as the deflections at ultimate loads.

The shear resistance was derived experimentally from the ultimate load of the specimen measured at failure. V_{frp} was obtained by subtracting the total shear resistance from that of the control specimens (V_c+V_s). The gain in shear resistance due to CFRP was calculated as a percentage of the shear capacity of the corresponding control specimens (gain = V_{frp} / (V_c+V_s)). V_c and V_s were calculated based on the results achieved from the control test specimens (S0-CON, S3-CON and S1-CON). The values provided in Table 2 are calculated based on the following assumptions implicitly admitted in the design guidelines: a) the shear resistance due to concrete is the same whether the beam is reinforced with transverse steel reinforcement or not and whether the beam is strengthened with CFRP or not; and b) the shear resistance due to steel is the same whether the beam is strengthened with CFRP or not.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Load at rupture (kN)</th>
<th>Shear resistance (kN)</th>
<th>( V_c ) (kN)</th>
<th>( V_s ) (kN)</th>
<th>( V_{frp} ) (kN)</th>
<th>Gain due to CFRP (%)</th>
<th>Maximum deflection (mm)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>S0-CON</td>
<td>127</td>
<td>84</td>
<td>84</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>2.6</td>
<td>Shear</td>
</tr>
<tr>
<td>S3-CON</td>
<td>284</td>
<td>188</td>
<td>84</td>
<td>104</td>
<td>0</td>
<td>0</td>
<td>11.3</td>
<td>Shear</td>
</tr>
<tr>
<td>S1-CON</td>
<td>355</td>
<td>235</td>
<td>84</td>
<td>151</td>
<td>0</td>
<td>0</td>
<td>11.9</td>
<td>Shear</td>
</tr>
<tr>
<td>S0-LS-RF</td>
<td>308</td>
<td>204</td>
<td>84</td>
<td>0</td>
<td>120</td>
<td>143</td>
<td>11.0</td>
<td>Shear</td>
</tr>
<tr>
<td>S3-LS-RF</td>
<td>416</td>
<td>275</td>
<td>84</td>
<td>104</td>
<td>87</td>
<td>47</td>
<td>18.0</td>
<td>Shear</td>
</tr>
<tr>
<td>S1-LS-RF</td>
<td>425</td>
<td>282</td>
<td>84</td>
<td>151</td>
<td>47</td>
<td>20</td>
<td>24.8</td>
<td>Shear</td>
</tr>
</tbody>
</table>

Cracking patterns and failure mode

All specimens exhibited a shear failure mode by crushing of concrete (S0 series) or yielding of steel-stirrups followed by concrete crushing and CFRP debonding (S3 and S1 specimens), as shown in Figure 3. Specimen S0-LS-RF ruptured with one principal diagonal shear crack, whereas distributed shear cracks formed around the main shear crack in the specimens with steel stirrups. Strengthened specimens with steel stirrups exhibited a considerable strain beyond the yielding point of longitudinal steel, particularly in specimen S1-LS-RF, allowing therefore a more ductile behavior of the shear-strengthened RC T-beams before reaching their ultimate failure.
The load corresponding to initiation and propagation of shear cracks was similar in all specimens; the cracks initiated at mid-height of the web and propagated towards the support and the load application point. The number of distributed shear cracks increased with the steel-stirrup ratio along the shear span, whereas thinner cracks were obtained accordingly. In specimen S0-LS-RF, the shear-crack angle was flat and almost connected the loading point to the support, with an angle of 28 degree. It became steeper in specimens S3-LS-RF and S1-LS-RF with 35 degree and 40 degree, respectively. With the presence of transverse steel, the crack propagated along the flange for a longer distance, causing steeper angles compared to specimens of S0 series.

No debonding or slippage of CFRP L-shape laminates was observed before reaching ultimate loads, indicating that a more effective utilization of the CFRP can be achieved when preventing debonding failure. This demonstrates the effectiveness of using the CFRP closed-stirrups in shear strengthening of RC T-beams.

![Figure 3 Typical shear failure mode](image)

(a) concrete crushing of anchored specimens  
(b) rupture of steel stirrups

**Gain in shear resistance due to CFRP**

As shown in Table 2, the shear resistance of S0-LS-RF reached 204 kN compared to 84 kN in S0-CON, which represents a 143% gain in capacity due to CFRP with respect to the control specimen. In the S3 series, with four steel stirrups provided along the shear span, the gain in shear resistance due to CFRP was 47% in S3-LS-RF with respect to S3-CON, whereas for S1 series, with six steel stirrups along the shear span, the gain was 20% in S1-LS-RF compared to S1-CON. This can be attributed to the anchorage provided by the CFRP rope that prevented premature debonding failure, demonstrating the effectiveness of using the CFRP closed-stirrups in increasing the shear capacity of shear-strengthened RC T-beams. In the other hand, a comparison between the stirrup series revealed a substantial reduction in the resistance gain due to CFRP with the increase in steel-stirrup ratio. This confirms the findings of other researchers on the inverse interaction between internal transverse steel reinforcement and externally bonded CFRP composites (e.g., Bousselham and Chaallal 2004, Chaallal et al. 2002, Pellegrino and Modena 2002), but not yet captured in major current international design codes, such as ACI 440.2R-08 or CSA S806-12.

**Deflection response**

From the results presented in Table 2, it is clear that deflections at ultimate loads increased significantly in the CFRP rope-anchored specimens with respect to their corresponding control counterparts: from 2.6 mm to 11 mm in S0 series; 11.3 mm to 18 mm in S3 series; and 11.9 mm to 24.8 mm in S1 series. Similar results were obtained with the increase in steel-stirrup ratio for both control and strengthened specimens. Neither specimen in the S0 series reached its flexural elastic limit, despite the considerable increase in shear resistance due to CFRP in S0-LS-RF. However, CFRP rope-anchored specimens with steel-stirrups (S3-LS-RF and S1-LS-RF) experienced a ductile behavior due to yielding of the longitudinal steel before ultimate failure by shear. Overall, the CFRP closed-stirrup strengthening technique greatly enhanced the ductility performance of RC T-beams.

**Strain response in CFRP**

The CFRP closed-stirrups greatly contributed to shear resistance. In fact, all strengthened specimens exhibited a significant increase in CFRP strains after reaching the shear capacities of their corresponding control counterparts: the increase in CFRP strain in S0-LS-RF was about 5300 μstrains (i.e., for a shear resistance exceeding 84 kN up to ultimate load at 204 kN); in specimens with steel stirrups, the increase was about 3150 μstrains in S3-LS-RF and about 1400 μstrains in S1-LS-RF under the same conditions. These results revealed a substantial reduction in
the CFRP contribution to shear resistance with the increase in steel-stirrup ratio along the shear span. The maximum CFRP strains reached at ultimate were achieved by S0-LS-RF with a value of 5400 \( \mu \)strains, which represents 42% of its ultimate strain. This demonstrates the effectiveness of using such a strengthening technique and, at the same time, confirms once again the existence of an interaction, and hence a stress redistribution, between the internal transverse steel and the EB-CFRP composites.

CONCLUSIONS

Within the experimental scope of this investigation, the following conclusions related to the use of closed-stirrups can be drawn:

1. The increase in shear resistance of strengthened specimens was about 70% in average; the increase in the specimen without steel stirrups reached 143%. This demonstrates the effectiveness of using the CFRP closed-stirrups in increasing the shear capacity of shear-strengthened RC T-beams.
2. The abrupt debonding failure mode was prevented in all strengthened specimens, allowing a more effective utilization of the CFRP strengthening material and resulting thereby in a much enhanced shear capacity.
3. A substantial reduction in the CFRP contribution to shear resistance occurred with the increase in steel-stirrup ratio, confirming the existence of an interaction between the internal transverse steel and the CFRP.

REFERENCES


AN EXPERIMENTAL INVESTIGATION ON THE SPLAY PORTION OF EMBEDDED FRP TENSION ANCHORS

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ABSTRACT

The use of fiber-reinforced polymers (FRP) for the repair and seismic retrofit of existing concrete and masonry structures is continually growing. An integral part of the FRP solution is the use of FRP anchors as anchors can delay unwanted premature debonding failure. Anchors can also help to transfer forces from the FRP to adjacent structural elements. FRP anchors comprise two main components, namely (i) a portion embedded into the concrete or masonry member, and (ii) a splay portion that is connected to the FRP strengthening. It is the splay portion of the anchors which is considered in this paper as there has been very little research conducted to date in this area. A series of tests is therefore reported and a design formula is proposed.

KEYWORDS

Design, Fiber Reinforced Polymers (FRP), FRP anchors, strengthening.

INTRODUCTION

An inherent limitation of the repair and seismic retrofit of existing concrete and masonry structures with fiber reinforced polymer (FRP) composites is the propensity of the FRP to debond. Debonding can occur in a sudden manner and at FRP strains well below the ultimate strain capacity of the material. The addition of anchorage devices (i.e. FRP anchors) is becoming an integral part of FRP repair and retrofit solutions. Figures 1 and 2 show typical schematic configurations of embedded FRP tension anchors (herein FRP anchors) at walls and columns. The two distinctly different constituents of the FRP anchors shown in the figures are (i) embedded portion of the anchor, and (ii) anchor splay (or anchor fan) portion. As is clear from the details that the two anchor components exhibit very different behaviours. The embedded portion of the anchor performs in tensile pullout similar to an adhesive anchor with failure modes such as concrete cone failure or anchor pullout. Related experimental investigations have been reported to date (Ozdemir and Akyuz 2005, Kim and Smith 2009, Ozbakkaloglu and Saatcioglu 2009) as well as a design approach (Kim and Smith 2010). The splayed portion of the FRP anchor, which helps develop force transfer, depends on the shear strength of the adhesive used to bond the splay to the FRP, as well as the splay configuration. Research concerning the splay portion is most limited.

The splay end of the anchor is where the force transfer between the FRP strengthening and the anchor occurs. The fibers at the free end of the anchors are distributed in a splay over FRP plates and adhered to the FRP using the same epoxy used to apply the FRP. Sometimes a ‘patch’ of either bidirectional or unidirectional FRP is placed over the splay, unless the splay is placed between consecutive layers of FRP. Although such configurations are commonly used in practice, little information is available for the design of such splay connections. It is commonly assumed that the design basis for splays is the shear bond capacity of the epoxy although this remains to be confirmed to any great certainty.

Kobayashi et al (2001) investigated a variety of splay lengths and splay angles (i.e. the angle included between the outer edges of the splay). Their experiments concluded that such splay connections could be effective in transferring force between the FRP and the anchor if the anchor had adequate FRP material to exceed the tensile capacity of the fabric being developed. Also, they found that installing a layer of perpendicular FRP over the splay was effective in mobilizing the splays over their full width. However, their testing utilised smaller diameter anchors (approximately 6 mm) than are typically being specified in practice. While it is possible to make general performance observations from the Kobayashi results, it is not appropriate to extrapolate them to design recommendations for larger diameter anchors which will necessarily involve greater tensile demands and larger
splay sizes. In an attempt to gain more knowledge on the behavior of such splay connections for larger diameter anchors, a test program was conducted by KL Structures. A part of the results of this program are reported herein in addition to a design recommendation.

Figure 1 Detail of a typical embedded tension anchor

Figure 2 Typical use of embedded tension anchors
**EXPERIMENTAL PROGRAM**

In current practice the splay area is designed on the basis of the shear bond capacity of the epoxy used for adhering the splay to the FRP strengthening. The objective of this test program was therefore to develop guidance on how much of this shear bond capacity could be used in design. The test assembly utilised is shown in Figure 3 while Figure 4 shows a test in progress as well as a typical test specimen. The test parameters are provided in Table 1. Six nominally identical tests were conducted for each specimen group ID. All tests failed in the splay region except for three tests in Group 2 that failed in the grip region. These three undesirable results are therefore excluded herein. The bond strength of FRP to concrete and masonry is almost always governed by the tensile strength of the substrate. Since the objective of this test program was to solely test the bond between the FRP and the anchor splay, it was decided to use steel plates as the substrate material. The steel plates were sandblasted and solvent-wiped prior to installation of the base FRP. Pull-off tests showed the adhesion between the specially prepared steel plates and the base FRP to be well in excess of the anticipated shear bond stress between the FRP and the anchor splays.

![Figure 3 Test setup for splay testing](image)

<table>
<thead>
<tr>
<th>Specimen group ID</th>
<th>Approximate Anchor Diameter (mm)</th>
<th>Splay Length (mm)</th>
<th>Splay Width (mm)</th>
<th>Bonded Area (cm²)</th>
<th>Equivalent 600 gsm Fabric Width (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>9.5</td>
<td>100</td>
<td>87.5</td>
<td>65</td>
<td>200</td>
</tr>
<tr>
<td>2</td>
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<td>150</td>
<td>125</td>
<td>126</td>
<td>375</td>
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<td>16</td>
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<td>125</td>
<td>168</td>
<td>563</td>
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</tbody>
</table>

Footnotes:  
1) Manufacturer-reported FRP laminate properties: ultimate tensile strength = 825MPa; laminate thickness = 1.27mm; tensile modulus = 69.6GPa.
2) Manufacturer-reported epoxy properties: tensile strength = 55MPa; tensile modulus = 2,755MPa; shear bond strength = 15.1MPa

Table 1 Splay test program

![Figure 4 Test Setup for splay testing and typical specimen](image)
The tests were performed in a universal testing machine, wherein the tension load was applied to the specimen via the loading head while the plate with FRP was anchored to the base via a steel bracket. The main results extracted from the tests are the total tensile load and the total cross-head displacement. A typical result, in terms of the load-displacement relationship, is shown in Figure 5, for the set of 100mm long splay tests (i.e. specimen group 1). A horizontal line is drawn to show the mean minus one standard deviation strength result. The measured splay strengths and calculated shear bond stresses, based on the mean minus one standard deviation values, of all there sets of tests are presented in Table 2.

<table>
<thead>
<tr>
<th>Specimen group ID</th>
<th>Splay Length (mm)</th>
<th>Bonded Area (cm$^2$)</th>
<th>Capacity based on Mean – 1.0SD (KN)</th>
<th>Shear bond based on test value (MPa)</th>
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<td>1</td>
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</table>

Table 2 Splay test results

It can be observed from Figure 5 that specimen 1 showed the lowest tension failure load. This particular specimen was dropped during fabrication resulting in the splay being separated from the base FRP prior to full cure of the epoxy. The FRP and the splays were allowed to cure, were mechanically abraded, solvent wiped and rebonded. The low tensile failure load is believed to be a result of this fabrication error. Specimen 2 exhibited a progressive debonding wherein the initial debonding was of an approximately 50mm wide strip of the base FRP from the steel plate followed by a partial debonding of the splay from the base FRP. Specimens 3 and 5 exhibited a similar behavior to that for specimen 2 with the exception that the failure of the base FRP from the plate was very limited. Specimen 4 failed by rupture in the neck region of the anchor, above the steel splay. This was attributed to inadequate saturation of the fibers which was observed in the failed region. Specimen 6 exhibited an almost complete shear failure between the splay and the FRP. Figure 6 shows a variety of failure modes observed in the tests, ranging from shear debonding of the splay from the base FRP (failure in the splay-FRP interface), to debonding of the splay and base FRP from the substrate (failure not just in the splay-FRP interface).
Figure 6 Different failure modes from splay testing

It can be seen from Figure 5 that there is significant variation in the maximum tensile strength, stiffness as well as the failure modes for the 100mm long splays. These were the first specimens fabricated for the tests and some of the inconsistency in the results may be a result of the evolving fabrication process which was well established for the larger splays. It was observed that for the 200mm long splays, the failure tended to either complete debonding of the splay from the FRP or complete debonding of the splay and base FRP from the steel plate. For the 150mm long splays the behavior was a mixture of that observed for the 100mm and 200mm long splays. This difference in behavior could be attributed to the inherent variability of the smaller splays which results from the smaller contact area between the splay and the FRP. As Table 2 clearly shows, the average failure shear bond stress decreased as the splay size increased but the failure mechanisms of the longer splay samples appear to indicated a more uniform distribution of the bond stress over the bonded area. The effect of a patch over the splay area was not studied in this effort, but it is believed that such a patch could only improve the behavior of the splay by distributing the forces across the splay and by providing local stability to the fibers.

During the testing, it was observed that the setup inherently created an eccentricity between the applied load and the resultant of the resistance at the base. Although small, this eccentricity created a peeling effect between the plate with the base FRP and the anchor splay, especially in the larger splay tests (See Figure 6(d)), and likely contributed to the lower tensile failure loads for the larger splay samples. It appears that the test may not have accurately simulated shear-only in the bonded area for all the specimens, because the tension/peeling force initiated a failure mode quite different from a pure shear condition. However, such eccentricities could exist when such anchors are installed in field conditions and so the tests are considered to still be appropriate for developing design guidance.

Figure 7 shows a plot of the measured bond stresses for all the different size splays. The lowest bond stress (corresponding to about 40% of the epoxy shear bond strength) was for a 200mm long splay where the failure was peeling off of the splay from the substrate FRP due to the eccentricity of the setup. A horizontal line, corresponding
to 35% of the shear bond strength of the epoxy, is drawn on the plot as a threshold value. It can be seen that all the measured values for bond stress are above this threshold.

Based on the above-described results of the splay tests, the following equation is proposed for splay capacity of the anchor:

$$F_S = 0.35V_{sb}A_S$$

where $V_{sb} =$ shear bond strength of epoxy as reported by manufacturer, $A_s =$ area of splay bonded to substrate FRP. This resulting bond stress value can be used to design anchor splays. All appropriate code specified strength reduction factors can be applied to the resulting splay capacity during design.

CONCLUSIONS

This paper has presented the details of an experimental program concerned with the splay portion of embedded FRP tension anchors. The specimens incorporated many of the fabrication issues associated with such anchors. In addition, a combination of shear and peeling stresses are simulated on the splay and this is representative of what can be experienced in the field. A design equation has been proposed based on the experimental results.

ACKNOWLEDGMENTS

KL Structures would like to thank Tim Ervin (Kulstoff Composite Products) for fabrication of the anchors as well as his invaluable assistance in the preparation of the specimens and the test setup.

REFERENCES

FORCE-BASED MODEL FOR STRAIGHT FRP ANCHORS EXHIBITING FIBRE RUPTURE FAILURE MODE

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ABSTRACT
The use of Fibre Reinforced Polymer (FRP) materials as Externally Bonded Reinforcements (EBR) is an established technique for structural improvement of existing buildings. Nevertheless, the technique features disadvantages, and premature FRP-to-concrete debonding has been commonly highlighted as one of the main problems, together with the difficulty to fully wrap the structural element when the structure presents complex geometries. FRP straight anchors are used to transfer the forces from the FRP sheet into the structural element, eliminating these two problems, but a comprehensive design method for FRP anchors has not yet been established despite the increased use and research on FRP anchors.
An extensive experimental programme has been carried out as part of an on-going research project with the ultimate goal being the development of a design methodology to enable engineers to efficiently and reliably design FRP anchors. The influence of a number of parameters on the capacity of straight FRP anchors has been investigated in the research, but only the anchor size and the fanning angle of the fan portion are reported here. The model that defines the relationship between anchor size, fanning angle and the capacity of the anchor exhibiting fibre rupture is described.

KEYWORDS
FRP, FRP anchor, EBR, fibre rupture, strengthening, retrofit, model

INTRODUCTION
Externally Bonded Fibre Reinforced Polymer Reinforcements (FRP-EBR) of Reinforced Concrete (RC) structures have been extensively investigated by the research community and widely used by practicing engineers. Two of the main problems of FRP-EBR systems reported in the literature and encountered by engineers are the premature FRP-to-concrete debonding (Chen & Teng 2001) and the presence of obstructions that prevent the RC structural member to be fully wrapped (Kim et al. 2009). Amongst the existing anchorage systems to overcome these problems FRP anchors have been highlighted in multiple occasions as the best solution, due mainly to the better properties of FRP materials when compared to more traditional materials such as steel, the high strength vs weight ratio and the better compatibility of FRP to FRP systems compared to FRP to other materials systems, as example given by Kalfat (2013). An FRP anchor consists of a bundle of fibres, or a rolled FRP sheet, that is soaked into epoxy resin before one end of the bundle (1) anchor dowel) is introduced into a hole that has been pre-drilled into the structural element and the other end is splayed into a fan shape ((2) fan component) and bonded with epoxy resin to the FRP sheet, with the fibres’ transition from the dowel to the fan through the small part of the anchor ((3) key portion), see Figure 19.

Figure 19 Straight anchor and main parts

The main problem for a wide implementation of FRP anchors is the absence of design guidelines that would enable engineers to reliably determine the anchor strength and predict its behaviour (Kim & Smith 2009; Kalfat et al. 2013).
Previous efforts have been undertaken to characterize the strength and behaviour of the anchor dowel, with a design model developed by Kim and Smith (2010) based on the failure modes related to the anchor dowel, but a thorough research has not yet been conducted on the behaviour of the key portion and the fan component. Reported herein are the results of 72 tests as part of an experimental campaign aimed to characterize the behaviour of the key and the fan components of straight anchors. Two main failure modes have been identified related to the dowel and fan components, fibre rupture and fan-to-sheet debonding respectively, with the tests reported here being focused on the fibre rupture failure mode, which all tests exhibited, and further research being continued in order to characterize the fan-to-sheet debonding failure mode.

TEST SET-UP, MATERIALS AND SPECIMEN PROPERTIES

An innovative test set-up unique for the on-going research programme was designed, with the main objective of replicating as close as possible a commonly used real application while aligning the applied load with the installed anchor and preventing induced moments or shear forces on the anchor, for more details of the test set-up the reader is referred to (del Rey Castillo et al. 2015). The applied tensile load was recorded with a load cell and displacements and strains were measured with the Digital Image Correlation (DIC) technique, but only loads are reported here.

The base material used to manufacture the FRP anchors was supplied as a bundle of fibres with a standard fibre content and the bundles were then combined and/or divided to manufacture anchors with different fibre contents. The manufacturer-specified material properties for the FRP products used in this research are given in Table 7. The properties are expressed as net-fibre laminate properties.

Concrete of two strengths were used but the influence of this property on the final strength of the anchor is negligible. The compressive and tensile strength reported in Table 8 of each concrete mix was determined using NZS 3112-2(1986).

| CFRP fabric 1 | 0.343 | - | 64.8 | 1241 | 1055 | 0.016 | 0.010 |
| CFRP fabric 2 | 0.331 | 75.7 | 68.1 | 968 | 833 | 0.013 | 0.011 |
| CFRP Anchor | 28 mm² | - | 230 | - | 2100 | - | 0.016 |

Table 7 Manufacturer-specified FRP material net-fibre properties

Concrete mechanical properties

<table>
<thead>
<tr>
<th>Characteristic ultimate compressive strength</th>
<th>Weak 1</th>
<th>Weak 2</th>
<th>Strong 1</th>
<th>Strong 2</th>
<th>Strong 3</th>
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</thead>
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<td>25.7 MPa</td>
<td>24.9 MPa</td>
<td>42.5 MPa</td>
<td>35.9 MPa</td>
<td>38.2 MPa</td>
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<td>CoV=0.16</td>
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</table>

Table 8 Concrete mechanical properties

Amongst the several parameters considered in the research two have been reported herein, the anchor size (how many bundles of fibres have been used to manufacture the anchor and the corresponding net cross section area) and the fanning angle α (angle from the middle to the end of the fan component, see Figure 19). The specimens and the parameters used in the construction of the anchors are listed in Table 9, with the names indicating the number of bundles used in the anchor assemble (0.5 to 6 bundles), which linearly correspond to the Net Cross Section Area in mm², and the intended fanning angle α (15, 27 and 60 degrees) at the time of the anchor construction. The intended angle did not always correspond with the final angle obtained, which was measured once the resin was cured and is reported in Table 9 as Fanning Angle α in Degrees. As can be observed in the table, anchors featuring 6 bundles and an angle α equal to 15 and 60 degrees were not tested, the reasons being the large strength of the 15 degrees 6-bundle anchors that prevented the specimen to be properly tested with the available equipment or the infrequently used angle of 60 degrees with big anchors.

RESULTS

The failure mode for all the tests reported herein is fibre Rupture of the anchor, either at the Key portion (Rk), at Middle length (Rm) or at the Top of the anchor (Rt) as shown in Figure 20. These three sub-failure modes did not have an influence on the final strength of the anchor but indicate that the fan-to-sheet debonding mechanism had
already commenced for the R\textsuperscript{m} and R\textsuperscript{t} modes. In 27 tests (37.5\% of 72 tests in total) this fibre rupture occurred after the FRP sheet had debonded from the concrete substrate, either completely (Complete Concrete Debonding or CCD) or partially (Partial Concrete Debonding or PCD) but the ultimate strength was not affected by the FRP-to-concrete debonding as already reported before, see (del Rey Castillo et al. 2015). Therefore 9 sub-Failure Modes (FM) exist and are reported in Table 10, R\textsuperscript{1} (40.3\% of 72 test), R\textsuperscript{m} (9.7\%), R\textsuperscript{t} (12.5\%), PCD+R\textsuperscript{m} (9.7\%), PCD+R\textsuperscript{t} (4.2\%), PCD+R\textsuperscript{1} (0.0\%), CCD+R\textsuperscript{m} (9.7\%), CCD+R\textsuperscript{t} (13.9\%) and CCD+R\textsuperscript{1} (0.0\%).

<table>
<thead>
<tr>
<th>Name</th>
<th>Net Cross Area (mm\textsuperscript{2})</th>
<th>Fanning Angle (\alpha) (DEG)</th>
<th>Name</th>
<th>Net Cross Area (mm\textsuperscript{2})</th>
<th>Fanning Angle (\alpha) (DEG)</th>
<th>Name</th>
<th>Net Cross Area (mm\textsuperscript{2})</th>
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</table>

The Ultimate Tensile Load (UL) at failure is reported in kN in

Table 10, together with the Real Strength (RS) in MPa based on the ultimate load and the anchor net cross section area. The Efficiency (Eff) was obtained as the percentage ratio of the Real Strength to the manufacturer-specified strength as given in Table 7. The manufacturer specifies the strength of one bundle so the manufacturer strength of multiple bundle anchors was linearly calculated by multiplying the manufacturer strength of one bundle by the number of bundles used to manufacture the anchor.
Figure 20 Three variations of fibre rupture failure mode

### Table 10: Test results

<table>
<thead>
<tr>
<th>Name</th>
<th>UTL$^1$ (kN)</th>
<th>RS$^2$ (MPa)</th>
<th>Eff$^3$ (%)</th>
<th>FM$^4$</th>
<th>Name</th>
<th>UTL$^1$ (kN)</th>
<th>RS$^2$ (MPa)</th>
<th>Eff$^3$ (%)</th>
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<td>PCD+R$^k$</td>
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<td>168.2</td>
<td>1001</td>
<td>47.7</td>
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</tbody>
</table>

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1Ultimate Load in kN, 2Real Strength in MPa, 3Efficiency in %, and 4Failure Mode, see above for more details

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FORCE TRANSFER MECHANISM AND TENSILE STRENGTH MODEL

The tensile force is transferred from the FRP sheet to the FRP anchor through the fan component and into the RC structure via the dowel, see Figure 21, with the force in the sheet being equal to the force in the dowel for the system to be in equilibrium, Eq 1a. By dividing the fan component into infinitesimal parts the infinitesimal force \( f_i \) is obtained, having this force two components being perpendicular and parallel to the force, as already hypothesized by Kobayashi (2001). Each of the parallel components can be calculated with the cosine of their corresponding fanning angle \( \alpha_i \), and the sum of all the \( f_i \) components equals the force transmitted to the dowel. Thus the Ultimate Load of the anchor exhibiting fibre rupture failure mode \( (N_{fr}) \) in N is equal to two times the cross sectional area in the dowel \((A_{dowel})\) multiplied by the tensile strength of the FRP \((S_{FRP})\) in MPa and by the sum of the parallel components of the force, reduced by a Efficiency Coefficient \( \psi_{eff} \), as shown in Eq 1b.

\[
\sum F = 0 \rightarrow F_{sheet} = F_{dowel} \tag{1a}
\]

\[
N_{fr} = \psi_{eff} S_{FRP} \sum \cos(\alpha_i) A_{dowel} \tag{1b}
\]

In order to calculate the sum of the \( \cos(\alpha_i) \) the integral between 0 and \( \alpha_0 \) (maximum angle \( \alpha \)) of the cosine function can be used, see Figure 22 and Eq 2a. However, the use of this equation would imply that for a fixed cross section area the quantity of fibres in the anchor increases as the angle increases. This behaviour does not represent the reality because the number of fibres remains constant for a constant cross sectional area although the fibres are less densely distributed in the anchor. To account for this effect an average value \( \alpha' \) has to be used, which is calculated by dividing the area by the maximum angle \( \alpha_0 \) and by substituting Eq 1b into equation 2b to obtain and expression for \( N_{fr} \).

\[
\int_0^{\alpha_0} \cos(\alpha) d\alpha = \left[ \sin(\alpha) \right]_0^{\alpha_0} = \sin(\alpha_0) - \sin(0) \rightarrow \alpha' = \frac{\sin(\alpha_0)}{\alpha_0} \rightarrow N_{fr} = \psi_{eff} S_{FRP} \frac{\sin(\alpha_0)}{\alpha_0} A_{dowel} \tag{2a}
\]

The efficiency coefficient for each test was then obtained dividing the ultimate load (UL) in Table 10 by the tensile strength \((S_{FRP})\) and the cross sectional area \((A_{dowel})\). This coefficient was then plotted against the angle \( \alpha \) for each anchor size, see Figure 23 (a) to (e) and the best fit obtained, see Eq 3 (a) to (e). The coefficients were correlated with their size as per Figure 23 (f) and the best fit again obtained for the efficiency factor as a function of the cross sectional area, see Eq 3f. Anchors featuring 6 bundles and an angle \( \alpha \) equal to 15 and 60 degrees were not tested, the reasons being the large strength of the 15 degrees six-bundle anchors that prevented the specimen to be properly tested with the available equipment and the infrequent use of angles of 60 degrees with such big anchors. The average efficiency

\[
\begin{align*}
\psi_{eff}^{14} &= 1.20 \frac{\sin(\alpha)}{\alpha} \tag{3a} \\
\psi_{eff}^{28} &= 0.99 \frac{\sin(\alpha)}{\alpha} \tag{3b} \\
\psi_{eff}^{42} &= 0.97 \frac{\sin(\alpha)}{\alpha} \tag{3c} \\
\psi_{eff}^{56} &= 0.80 \frac{\sin(\alpha)}{\alpha} \tag{3d} \\
\psi_{eff}^{84} &= 0.75 \frac{\sin(\alpha)}{\alpha} \tag{3e} \\
\psi_{dowel}^{14} &= 2.41 A_{dowel}^{-0.26} \tag{3f}
\end{align*}
\]
coefficient (0.4) for these tests was used in equation 3f.

Figure 23 Calibration of theoretical model with experimental results
For consistency with the equation described in Eq 1b the inverse of the cosine is applied to obtain a cosine function, Eq 4a, but for simplicity $0.57\alpha_0$ can be used for the range of angles under study (0 to 60 degrees), see Figure 24. By substituting this relationship into Eq 1b the governing equation 4b for $N_f$ is obtained, and by solving $y$ for Eq 3f the final equation is obtained, Eq 4c. The three dimensional surface describing Eq 4c can be seen in Figure 26, together with the experimental data points. For the biggest anchors made with six bundles the equation overestimated the strength and further research is needed in order to adjust these values. The influence of the fanning angle on the total capacity of the anchor is larger as the anchor increases its size and anchors with a more narrow angle $\alpha$ are more efficient than anchors with a more obtuse angle $\alpha$.

\[
\alpha' = \frac{\sin(\alpha_0)}{\alpha_0} \rightarrow \cos^{-1}(\alpha') = \cos^{-1}\left(\frac{\sin(\alpha_0)}{\alpha_0}\right) \equiv \cos(0.57\alpha_0)
\]

\[
\sum_i \cos(\alpha_i) = \cos(0.57\alpha_0) \rightarrow N_{fr} = \psi_{eff} \sigma_{FRP} \cos(0.57\alpha_0) A_{dowel}^{2/3}
\]

In order to obtain the lower bound equation for use in design the standard deviation was analysed for each group of data points with the same anchor size and fanning angle, but no trend was found within each anchor size, see Figure 25a. The standard deviation was then studied for the different anchor sizes, and, as can be observed in Figure 25b, it was established that the standard deviation increases as the anchors increased in size, following a power equation. By subtracting the standard deviation from the model the lower bound model can be found, see equation 4d, with the representation of this model being reported in Figure 26 together with the data from the experimental results. The ultimate force in kN as predicted by the model for each specimen is represented with a square and the recorded force is represented with a circle, with the difference between the two forces being represented with a solid line. As can be observed in Figure 26 the influence of the fanning angle on the total capacity of the anchor is larger as the anchor increases in size, and anchors with a narrower angle $\alpha$ are more efficient than anchors with a more obtuse angle $\alpha$. 

(a) Standard Deviation for each anchor size

(b) Standard Deviation for the test population
CONCLUSION AND FUTURE WORKS

Reported herein are the results of 72 tests on CFRP straight anchors that exhibited fibre rupture failure mode with varying anchor size and fanning angle $\alpha$. The sub-failure mode that the anchor exhibited did not affect the strength of the anchor, as long as the anchor could resist the energy released when the FRP-to-concrete debonding occurred. Anchors featuring an acute fanning angle generally behave better than anchors with a more obtuse angle, but this influence is almost negligible for small anchors and becomes more critical as the anchor size increases. Only ultimate loads are reported here, although strains and displacements were measured using the DIC technique. The force transfer mechanism between sheet, anchor and concrete substrate is analysed and a number of equations for different anchor sizes with a varying fanning angle $\alpha$ are reported (Eq 3a to f). A final equation and the corresponding lower bound equation are described, taking into consideration both anchor size and angle $\alpha$ to help engineers to calculate the final strength and anticipate the behaviour of straight CFRP anchors in a reliable way. Practicing engineers now have a tool to predict the maximum load that an FRP anchor is able to withstand for a given size and fanning angle $\alpha$.

However, the behaviour of the anchor is not yet fully understood, especially for big anchors that are difficult to test. A numerical model will help to understand the fracture mechanism and the influence of fanning angle $\alpha$ on the final strength of the anchor. A sensitivity analysis can be carried out to consider anchors with larger cross sectional areas and to increase the range of FRP and epoxy resin properties under study. Finally, a real case study application will be tested in order to verify the validity of the equations and investigate the influence of tensile-compression cycles on the anchor strength and behaviour.

ACKNOWLEDGEMENTS

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REFERENCES
ABSTRACT

Recent studies have shown the efficiency of the application of Carbon Fibre Reinforced Polymer (CFRP) materials to reinforce cracked steel elements. Bonding CFRP materials on cracked steel plates leads to a significant reduction of the stress intensity factor (SIF) at the crack tip and thus to a significant increase of the fatigue life. This paper deals with the reinforcement of old steel plates (mild steel and puddled/wrought iron) by adhesively bonding CFRP laminates with Normal and Ultra High Modulus (NM and UHM, respectively). The chosen steel plate geometry is consistent with the one of old riveted connections. Steel plates with a single crack emanating from a centre hole are studied. Symmetrical and un-symmetrical reinforcement are considered as well as pre-stressing before application. Experimental results, showing efficiency of the different studied reinforcement configurations, are then used to determine the modified mode I SIF. Such a result could then be used in reinforcement operations to assess the provided increase of life expectancy.

KEYWORDS
CFRP, fatigue, old steel structures, stress intensity factor.

INTRODUCTION

The preservation of historic metallic bridges is today a major challenge. The major causes of deterioration of these old structures are corrosion and fatigue phenomenon. Moreover, particular typologies of damage are recognized in these structures due to the used materials (wrought iron and mild steel) and the riveted connections. The traditional method of strengthening is based on the application of steel plates to the original structures that are welded, bolted or riveted. This traditional method shows some negative effects as the uses of welded, bolted or riveted plates may create residual stress and induce new stress concentration areas that are prone to fatigue. Moreover, the additional steel plates increase the permanent loads to the weakened structure and are prone to the same phenomena of corrosion and fatigue.

The use of CFRP in civil engineering is more developed in the case of concrete structures even if, for over ten years, the researchers have developed a greater interest for the case of metallic structures. Studies conducted on metallic structures reinforced by CFRP bonded have shown the possibility of enhancing the performance of structural members: increase of carrying capacity, stiffness, ductility, durability and fatigue resistance (Zhao et al. 2014). In addition, CFRP materials show some advantages in comparison to steel, such as high strength and rigidity for a low weight, no corrosion sensitivity and fatigue resistance. Moreover, FRP materials are easy to handle and their use in strengthening intervention may need shorter time. Consequently, it was decided with the collaboration of the French Railway Company to study the possibilities offered by such technology for the specific case of old riveted metallic assemblies. Previous studies consisted in analysing the behaviour of the riveted assembly (Lepretre et al. 2014) and the adhesively bonded joint (Chataigner et al. 2012; Lepretre et al. 2016). It was then decided to carry out investigations regarding the ability to increase life expectancy of cracked steel assemblies.

To investigate the efficiency of CFRP-reinforced steel elements in fatigue crack-growth behaviour, some research studies were led on various cracked steel plates configurations and for different CFRP materials. Liu et al. (2009) also studied the repair of cracked steel plates with centre hole by bonding NM CFRP sheets. The influence of some parameters such as bond length, bond width, double and single side repair was investigated. The fatigue life was increased for all reinforced specimens, and all the more in the case of double side repair. The experimental results showed that fatigue life could be extended by up to 7.9 times compared to the un-repaired specimens. Wu et al. (2012) adopted similar test specimen’s geometry to test Ultra High Modulus (UHM) CFRP laminates.
reinforcement. Five strengthening configurations with single and double side repair were investigated. The results show an increase of fatigue life by a factor ranging from 3.26 to 7.47.

Some researchers also investigated the pre-stressing technique of NM CFRP laminates (Bassetti et al. 2000; Colombi et al. 2003; Täljsten et al. 2009; Ye et al. 2010). Bassetti et al. (2000) and Colombi et al. (2003) used NM CFRP laminates (E=210 GPa and e= 1.2mm) pre-stressed up to 41.2 kN to reinforce steel plates with centre hole. An increase of 16 times the fatigue life was obtained for double side repair.

Thus, increasing the Young’s modulus or pre-stressing NM CFRP plates give the best results in terms of fatigue life extension of cracked steel elements. Nevertheless, today only few studies have investigated these two solutions. In this paper, non-pre-stressed and pre-stressed NM CFRP laminates as well as UHM CFRP laminates were used to reinforce metallic plates with one fatigue crack emanating from a centre hole. Single and double side repair and different initial crack lengths were studied. Two metallic materials were used for the metallic elements: S235 carbon steel and wrought iron. Experimental results, showing efficiency of the different studied reinforcement configurations, are then used to determine the modified mode I SIF.

EXPERIMENTAL STUDY

Experimental study on metallic plates with various degrees of initial damage has been conducted in order to investigate the efficiency of the different reinforcement process on fatigue crack-propagation. Two types of metallic materials were used as well as two types of CFRP reinforcement process.

Used materials and sample geometry

S235 carbon steel and wrought iron materials were used for the metallic plates. S235 carbon steel grade was chosen due to its mechanical behaviour similar to that of mild steel used in the metallic bridges of the first half of the 20th century (Bassetti et al. 2000). Wrought iron material originates from the dismantling of an old riveted bridge. Tensile test were carried out to assess the mechanical properties of the materials. The main yield stress, tensile strength and Young’s modulus of the steel and wrought iron (in rolling direction) plates are indicated in Table 1. Two types of laminates with their associated adhesive were selected. Reinforcement process A uses Normal Modulus (NM) CFRP with linear elastic adhesive, while reinforcement process B uses Ultra High Modulus (UHM) CFRP with no linear elasto-plastic adhesive. The selected materials for the two reinforcement processes and some of their mechanical properties are listed in Table 1.

<p>| Table 1 | Measured properties of metallic, CFRP and adhesive materials |
|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|</p>
<table>
<thead>
<tr>
<th>S235 carbon steel</th>
<th>Wrought iron</th>
<th>NM CFRP laminate</th>
<th>UHM CFRP laminate</th>
<th>Adhesive A</th>
<th>Adhesive B</th>
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<td>506 (3.4)</td>
<td>306 (5.3)</td>
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<td>-</td>
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<td>Yield strength (MPa)</td>
<td>250 (14.5)</td>
<td>173 (5.7)</td>
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<td>(2.96)</td>
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<td>Tensile Modulus (GPa)</td>
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<td>187 500 (4120)</td>
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<td>460 000</td>
<td>3.65</td>
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<td>Thickness (mm)</td>
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<td>1.2</td>
<td>2.3</td>
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The values correspond to an average values from three test results, the standard deviation is indicated in brackets.

Specimens consist in metallic plates reinforced with CFRP bonded on one side or both sides. The carbon steel plates are 510 mm-long, 90 mm-wide and 10 mm-thick with a 20 mm-diameter hole and one initial 0.6 mm-wide notch emerging from the hole, as shown in Figure 1. For wrought iron plates, the dimensions are the same except the thickness of 7 mm of the metallic plates. The geometry of the plates was chosen to represent a part of a riveted joint. Thus, the hole’s diameter represents the rivet hole and the thickness of the plates are convenient with those of old steel bridges members. The initial notch was made using the wire erosion technique. This allows to localize the crack initiation and to obtain a straight crack through thickness of the plates. From this initial notch, a fatigue crack is then initiated and propagated until a given length corresponding to the initial crack length a0 for which the plate is reinforced by bonding CFRP laminates.
Two different initial crack lengths were chosen for this study to simulate various stages of crack propagation when the strengthening is applied: \( a_0 = 6 \text{ mm} \) and \( a_0 = 13 \text{ mm} \). The first chosen length corresponds to a crack under rivet head, only detected using no destructive control technique. While the second initial length of the crack corresponds to a crack that has exceeded the rivet head and that can be detected by visual examination.

For all reinforced specimens, the CFRP plate is bonded on one side or both sides of the cracked plate and at a certain distance from the edge of the hole, taking into account the presence of the rivet head. For this reason, all CFRP plates were bonded at a distance of 10 mm from the edge of the hole. Geometry of repaired plates, initial crack length and repaired configuration are shown in Figure 1.

The specimens were tested under tensile cyclic loading with a stress ratio of 0.1 and a frequency of 10Hz, until complete failure of the specimens. The applied stress in the nominal section of the plates is similar for both carbon steel and wrought iron plates and varies from 100 MPa to 10 MPa.

Crack propagation for all S235 carbon steel specimens was recorded using the “beach marking” technique. The principle of this technique is to apply a reduced stress range for a short number of cycles during the propagation stage of the crack. The modification of applied stress range decreases crack growth rate and creates visible marks on the crack surface. It is thus possible to observe the real crack size and shape in the thickness of the steel specimens after complete failure of them and to relate it to the load history. For the wrought iron specimens, the “beach marking” technique cannot be applied due to the metallurgy of the materials. In fact, due to the manufacturing process, the microstructure is non homogeneus with a lot of inclusions of sulphides and oxides which have a brittle behaviour (Pipinato et al. 2012). This led to anisotropy of the material especially bad in the thickness direction due to the arrangement of the inclusions and the influence of the rolling process. After several tests, it was decided to adopt a specific crack propagation sensor bonded on the surface to record the crack length with time.

**EXPERIMENTAL RESULTS**

Fatigue life and failure modes

The tests results are summarized in Table 2 for carbon steel specimens and Table 3 for wrought iron specimens. Three identical specimens were done for each reinforcement configuration. For S235 carbon steel plates, the experimental results show good repeatability. Thus, the results indicated in Table 2 are an average value obtained for the three testing coupons corresponding to each reinforcement configuration. For wrought iron materials only UHM CFRP reinforcement was investigated. Moreover, the non-homogeneity of the materials leads to different results for testing specimens with the same reinforcement configuration. For this reason, the results indicated in Table 3 are for only one specimen labelled with “-1/-2/-3”.
The fatigue cycle number is counted until complete failure of the specimen and the fatigue life increase ratio corresponds to the ratio between the average fatigue life of CFRP-reinforced specimens and those of the reference specimens (un-reinforced).

### Table 2: Test results for S235 carbon steel specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Initial crack length (notch + fatigue crack)</th>
<th>Reinforcement process</th>
<th>Reinforcement configuration</th>
<th>Fatigue cycle number</th>
<th>Fatigue life increase ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference</td>
<td>/</td>
<td>/</td>
<td>/</td>
<td>416 718</td>
<td>/</td>
</tr>
<tr>
<td>MN_NT_NS_A1</td>
<td>6</td>
<td>MN</td>
<td>SS + NP</td>
<td>538 843</td>
<td>1.29</td>
</tr>
<tr>
<td>MN_NT_NS_A2</td>
<td>13</td>
<td>MN</td>
<td>SS + NP</td>
<td>520 186</td>
<td>1.25</td>
</tr>
<tr>
<td>MN_T_NS_A1</td>
<td>6</td>
<td>MN</td>
<td>SS + P (10kN)</td>
<td>725 951</td>
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<td>MN</td>
<td>DS + NP</td>
<td>999 199</td>
<td>2.4</td>
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<tr>
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<td>MN</td>
<td>DS + NP</td>
<td>832 107</td>
<td>2</td>
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<tr>
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<td>UHM</td>
<td>SS + NP</td>
<td>979 127</td>
<td>2.35</td>
</tr>
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<td>SS + NP</td>
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<td>1.61</td>
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<td>983 593</td>
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<tr>
<td>UHM_S_A2</td>
<td>13</td>
<td>UHM</td>
<td>DS + NP</td>
<td>874 481</td>
<td>/</td>
</tr>
</tbody>
</table>

SS = single side repair  
DS = double side repair  
NP = non pre-stressed  
P = pre-stressed

### Table 3: Test results for wrought iron specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Initial crack length (notch + fatigue crack)</th>
<th>Reinforcement process</th>
<th>Reinforcement configuration</th>
<th>Fatigue cycle number</th>
<th>Fatigue life increase ratio</th>
</tr>
</thead>
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<tr>
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<tr>
<td>Reference 2</td>
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<td>546 497</td>
<td>/</td>
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<tr>
<td>F_UHM_NS_A1-1</td>
<td>7</td>
<td>UHM</td>
<td>SS + NP</td>
<td>1 037 170</td>
<td>2.12</td>
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<tr>
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<td>UHM</td>
<td>SS + NP</td>
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<td>SS + NP</td>
<td>1 877 567</td>
<td>3.85</td>
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</table>

Failure of all NM CFRP-reinforced specimens was caused by cohesive failure in the adhesive layer (see Figure 2a). While cohesive failure follows by delamination of the composite is observed for UHM CFRP-reinforced specimens whatever the metallic materials (see Figure 2b).

The results indicated in Table 2 for S235 carbon steel specimens show an increase of fatigue life by a factor ranging from 1.29 to 2.4. The maximum increase of lifetime is obtained for double side repair specimens. UHM_NT_S_A1 and UHM_NT_S_A2 fatigue life increase ratio for S235 steel plates were not indicated in Table 2. For this specimen’s reinforcement configuration, an applied stress range of 90 MPa shows a too slow propagation of the crack (no failure of the specimens after 3 million cycles). For this reason, the applied stress range was increased to 130 MPa, which does not allow a comparison in terms of fatigue cycle number with the others specimens.

In practice, it is often difficult to make a double side repairing due to the complex geometry of the cracked elements and more particularly for old riveted structures, which have angle iron elements and a lot of rivets. For single side repaired S235 carbon steel specimens, the maximum increase of lifetime is obtained for UHM CFRP reinforcement. This is why only this CFRP reinforcement was adopted for cracked wrought iron plates (see Table 3).
The non-repeatability of the tests for wrought iron specimens is clearly visible in Table 3. Nevertheless, for all CFRP-reinforced specimens the fatigue life increase ratio is significant with an increase lifetime ratio ranging from 2.12 to 3.85.

**Crack propagation curve**

As mentioned above, the evolution of the crack front shape during crack propagation for S235 carbon steel plates is determined using beach marking technique. The microscope observation of the crack surfaces is represented in Figure 3 for different reinforcement configuration.

![Crack propagation curves](image)

It is interesting to note that, for single-side UHM plate reinforcement, a corner crack appears at the other edge of the hole before complete failure of the specimens. That is not the case for single-side NM plate reinforcement. For double-side repair with NM and UHM plate, a corner crack also appears at the other edge of the hole for both cases. As expected, more striations are visible for CFRP-reinforced specimens depending on the reinforcement configuration. Two kinds of crack shapes, symmetric and un-symmetric about the mid-thickness axis, are observed. For un-repaired and double-side repaired specimens, the crack shape tends to be symmetric about the mid-thickness of the plate with the central point at the crack front (the maximum stress intensity factor is obtained for this central point in that case), see Figure 3a. While for un-symmetric specimens, the maximum stress intensity factor is obtained for the un-repaired surface intersection (see Figure 3b and 3c).

![Crack gauge images](image)

Figure 4c shows the crack surface of the wrought iron specimens. The brittle slag/inclusions and the non-homogeneity of the materials are clearly visible. The “beach” marking technique cannot be used for this material, so the crack propagation was followed by use of crack gauge (see Figure 4a and 4b). The crack length against cycle number is drawn in Figure 5 and Figure 6 for S235 carbon steel specimens and wrought iron specimens, respectively.
For all CFRP-reinforced S235 carbon steel plates, the reinforced specimens show a significant decrease of the crack propagation rate. Reinforced as early as possible and using UHM CFRP reinforcement allow a bigger increase fatigue lifetime. In all cases, the effect of CFRP laminate is maximal when the crack propagates under the composite laminate until a certain length after which the propagation of the fatigue crack becomes unstable. As for S235 carbon steel specimens, the CFRP-reinforced specimens show a significant decrease of the crack propagation rate even if fatigue test results show a wide dispersion. This is due to the non-homogeneity of the materials and because of the many inclusions.

In literature studies, the most investigated reinforcement configuration for cracked steel plates with centre notch (hole or crack) concerned the bonding of CFRP materials on all the cracked surfaces. In that case, the best fatigue strengthening effect is achieved. Liu et al. (2009), Wu et al. (2012) and Täljsten et al. (2009) have studied different reinforcement configuration with CFRP materials which do not cover the cracked surface (see Figure 7). In these studies, CFRP sheets and laminates are bonded on each side on the center crack and for different distance to it.

Liu et al. (2009) studied 5 layers MBrace CF350 sheets (Young’s Modulus of fibers is 552 GPa and equivalent modulus of the composite is 128 GPa) of 30mm-wide bonded on each side of the 5mm-diameter center hole and for single and double repair configuration. As expected, the single side repair configuration is less effective in
extending the fatigue life comparing with double side repair configuration. The fatigue life increase ratio was of 1.4 for single side repair and of 5 for double side repair specimens. Wu et al. (2012) show the clear influence of CFRP bonding locations on the fatigue behaviour studying two types of UHM CFRP laminates (E=477 GPa) locations on each side of the centre hole in steel plates and for double side repair configuration. Specimens with the longest distance of CFRP plates away from the crack tip yield the lowest strengthening efficiency with a fatigue life increase ratio of 3.26 compared with 5,82 for the other configuration. Täljsten et al. (2009) used two types of CFRP laminates for his study (E=155 GPa and E=260 GPa) pre-stressed and no pre-stressed with bigger wide of 50 mm. All the specimens tested were double side repair and the obtained results show an increase fatigue life ratio >>34 for specimen with CFRP laminate (E=260 GPa) pre-stressed to 12kN and >8 for specimen with CFRP laminate (E=155 GPa) pre-stressed to 15kN. The same specimens no pre-stressed show an increase fatigue life of 3.7 and 2.8 respectively.

In our case, the cracked specimens are un-symmetric (one crack emanating from the centre hole and reinforced only on the side of the crack) which leads to lower increase of the fatigue life for reinforced specimens. Nevertheless, as for the literature studies, the double side repair configuration and the use of UHM CFRP laminates are more effective in extending the fatigue life of cracked metallic plates.

### ASSESSMENT OF STRESS INTENSITY FACTOR FOR REPAIRED STEEL PLATE

The Stress Intensity Factor (SIF) is a key factor of the LEFM (linear elastic fracture mechanics). It is used to predict the crack propagation rate and thus assess the fatigue life of a cracked element. The SIF at the crack tip can be simply expressed in terms of the applied stress range, Δσ, the crack length, a, and a geometry correction factor, $F(a)$ (Albrecht and Yamada 1977):

$$\Delta K = F(a)\Delta\sigma\sqrt{a}$$  \hspace{1cm} (1)

Classical solutions of SIF values are available in literature for simple geometries of cracked elements (Tada et al. 2000, JSSC 1995, Albrecht and Yamada 1977). For the centre cracked steel plate with one crack emanating from the edge of the hole, Bowie et al. (1956) model with Newman modification (1976) (taking into account the finite width of the plate) can be adopted (Eq. 2).

$$F(a) = \varphi_1 \cdot f_1$$  \hspace{1cm} (2a)

$$\varphi_1 = 0.707 - 0.18X + 6.55X^2 - 10.54X^3 + 6.85X^4$$ \hspace{1cm} (Bowie solution)  \hspace{1cm} (2b)

$$X = \left(1 + \frac{a}{R}\right)^{-1}$$

$$f_1 = \left[\sec\left(\frac{\pi(2R + a)}{2(W - a)}\right)\right] \cdot \sec\left(\frac{\pi R}{W}\right)$$ \hspace{1cm} (Newman modification)  \hspace{1cm} (2c)

CFRP reinforcement will act in two ways on the SIF values. First of all, the CFRP laminate help to share the load applied to the cracked plate by transferring through the adhesive layer. As the crack propagates, more fatigue load can be taken by the CFRP laminates reducing the effective stress in the cracked steel plate. Secondly, the bonded CFRP laminates modified the geometry of the cracked element and then the geometry correction factor.

From the crack propagation curves obtained experimentally (cf Figure 5 and Figure 6), the experimental SIF can be calculated directly using the Paris law (Eq. 3) (Paris et Erdogan 1963). The "secant method" given in ASTM E647 (2001) was used for the determination of the crack growth rate da/dN (Eq. 4).

$$\left(\Delta K_{exp}\right)_u = \left[\frac{1}{C} \left(\frac{da}{dN}\right)_u\right]^{1/\nu}$$  \hspace{1cm} (3)

$$\left(\frac{da}{dN}\right)_u = \frac{a_{i+1} - a_i}{N_{i+1} - N_i}$$  \hspace{1cm} (4)

(da/dN)_u is the average crack growth rate over the crack length increment $a_{i+1} - a_i$ measured at the mid-thickness of the plates whatever the reinforcement configuration. The calculated values is for a crack length $\bar{a} = \frac{1}{2}(a_{i+1} - a_i)$. $C$ and $m$ are material constants determined experimentally from un-repaired reference specimens. Their values are 1.45.10^{-13} and 2.97 respectively for S235 carbon steel plates and 1.93.10^{-20} and 5.34 respectively for wrought iron plates (da/dN in mm/cycle and ΔK in MPa√m). Barsom (1974) suggests for mild steels: $C = 2.2.10^{-13}$ and $m = 3$. Another suggestion for old steels was given in ECCS (2005) and by Sedlacek et al. (1992): $C = 4.10^{-13}$ and $m = 3$ (values refer to upper range). Kühn (2013) has led a statistical analysis derived from an amount of 205 tests and has determined maximum and minimum values of Paris parameter for wrought iron materials: $C_{\text{min}} = 4.9.10^{-17}$ (R=0.1), $C_{\text{min}} = 5.7.10^{-28}$ (R=0.5) and $m_{\text{max}} = 9.3$ (R=0.5), $m_{\text{max}} = 3.8$ (R=0.5). Thus, the obtained results for both S235 carbon steel plates and wrought iron plates seem to be consistent with the compared literature studies. For
wrought iron it is recommended to apply the parameters referring to old steels. These parameters seem to be conservative but that allows compensating the high scatter of material behaviour. The SIF analysis is only done for S235 carbon steel specimens, which show a good repeatability and thus allow comparison between each reinforcement configuration. By plotting the stress intensity factor as a function of the crack length, the effectiveness of the reinforcement process can be assessed. SIF values will depend on the crack propagation stage, as ever mentioned by Wu et al. (2013b) who identified three stages of Mode I SIF with crack propagation when the CFRP is bonded away from the cracked area.

Figure 8 shows the SIF as a function of the crack length for repaired and un-repaired S235 carbon steel specimens.

![Figure 8 SIF against crack length for S235 specimens](image)

The SIF increases with the crack growth but it is different for the repaired specimens. Figure 8a shows the SIF for single side repair case with an initial crack length of 6 mm. It is clearly visible that UHM CFRP reinforcement leads to a more stable propagation of the crack (SIF values are stable for a longer crack length) and a higher SIF reduction. This last point was also noted by Wu et al. (2013a), who showed that the SIF of CCT steel plate depends on the relative stiffness, concluding that a higher stiffness resulting in greater reduction in SIF. Then, by comparing single side and double side repair cases (Figure 8a and b), we can see that, for UHM and NM pre-stressed CFRP single side reinforcement, the decrease of the SIF values happens before the crack has reached the edge of the CFRP plate. That is not the case for double side repair (see Figure 8b) for which the effective SIF decrease appears when the crack propagates under the CFRP laminate. The single side repair geometry configuration can explain this phenomenon as well as the fact that measures of crack length increment is done at mid-thickness of the specimens (un-symmetric crack shape, see Figure 3). For single side repair configuration, Figure 8c shows that, at the same crack length, SIF values are smaller for specimens with the smallest initial crack length. This phenomenon is more visible for UHM CFRP reinforced specimens.

CONCLUSIONS

In this paper, NM CFRP laminates and UHM CFR laminates were used to extend the fatigue life of cracked metallic plates with centre hole and one crack emanating from the hole. The pre-stressing technique of NM CFRP laminates was also investigated and double side and single side repair were done.

In order to assess the effectiveness of the CFRP reinforcement for strengthening old metallic structures, two types of metallic materials were used for the plates: mild steel and wrought iron. From the experimental results, it is evident that the fatigue behaviour of the cracked metallic plates is greatly improved by CFRP bonding. The results show an increase of fatigue life by a factor ranging from 1.25 to 2.4 for 235 carbon steel plates with different reinforcement configuration. The UHM CFRP reinforcement gave the best lifetime increase and was thus chosen for reinforcing wrought iron plates. Particular chemical and metallurgical characteristics of wrought iron lead to a
wide scatter of results for crack growth rate with the same repaired and un-repaired configuration. The “beach marking” technique cannot be used with this kind of materials (lot of inclusions) and the fatigue crack growth was thus being followed using crack gauge.

SIF evolution for each reinforced S235 carbon steel specimens was investigated in order to assess the efficiency of the reinforcement configuration. As expected, UHM CFRP reinforcement shows best results. For all cases, the repair efficiency is bigger when the initial crack length is smaller, as shown by Wang et al. (2013) and Yu et al. (2013). The next step of this study is to determine an empirical and numerical reduction factor for the SIF taking into account the particular geometry of the plates and of the reinforcement configuration. This type of study was ever done by Wu et al. (2013a) and (2013b) for centre-crack tensile steel plate (two symmetric cracks at hole edge) reinforced by CFRP bonded. In these studies, the authors showed the effects of CFRP bond location on the Mode I SIF and proposed a new correction factor as a function of crack length and bond location using both experimental and finite element models. Thus, it will be interesting in further work to adapt the methodology of these two studies to the case of a plate with only one crack at the edge of the hole and reinforced with only one CFRP laminate bonded in front of the crack tip. The final goal remains to determine a reduction factor for SIF for riveted assemblies reinforced by CFRP bonded.

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REFERENCES


DEVELOPMENT OF REPAIR AND STRENGTHENING METHOD OF STEEL MEMBERS USING VARTM TECHNOLOGY

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ABSTRACT

The objective in this study is to develop a repair and strengthening method of existing steel members by Vacuum assisted Resin Transfer Molding (VaRTM) technology. First, coupon specimens of hybrid FRP plates were fabricated using VaRTM and were tested in order to investigate fundamental material properties. Next, in order to study repair effect and bonding strength, steel plates bonded the hybrid FRP plates using VaRTM were tested under bending load. The results show the quality of material properties of hybrid FRP plates are high and equivalent to a pultruded material. Although the bonding strength varied widely, the bonding strength was high in general. Therefore, the significant repair effect by the VaRTM technology was confirmed.

KEYWORDS

VaRTM, hybrid composite FRP, steel member, repair and strengthening, bonded joint.

INTRODUCTION

FRPs have been recognized as a innovative structural material for an efficient rehabilitation and maintenance of infrastructures, because they are lightweight and advantageous for long-term durability. CFRPs, which are composed of carbon fibers, have been feasible for strengthening and repair of existing steel structures in practical use, so that they are high modulus, high strength, and easy to use in situ (Zhao 2013). Since CFRP strips are solid plates, a FRP bonding system is not advantageous to a discontinuous surface like a gap between members, connections and welded joints. Carbon fiber sheets are avetable to a discontinuous surface, however many layers of carbon fiber sheets are required because of a thin sheet and they increase construction period and construction cost. Therefore, the FRP bonding system for the construction of various situations is required.

Vacuum assisted Resin Transfer Molding (VaRTM) technology, which is available to fabrication of structural elements of aircrafts and blades of wind turbine, the objective of this study is the development of construction method for molding and bonding FRP to existing steel structures. This technology has been applied to strengthening of concrete structure (Uddin et al. 2013), not yet to steel structures. Figure 1 shows the schematic view of VaRTM technology.

Figure 1 Schematic view of VaRTM technology
First, in order to investigate fundamental material properties fabricated using VaRTM, coupon specimens of hybrid FRP (HFRP) plates which were composed of carbon fiber (CF) and glass fiber (GF), were tested. Next, in order to study the effect of strengthening and bonding strength, the steel plates bonded the hybrid FRP plates using VaRTM were tested under bending load.

**MATERIAL PROPERTIES OF HFRP PLATES FABRICATED BY VaRTM**

**Design of Test Specimens and Experimental Program**

*MATERIAL configuration and matrix resin*

The HFRP plates which were composed of CF and GF, were selected as reinforcement materials for steel structures. Table 1 shows the laminate configuration. The design equivalent elastic modulus was set to 60GPa, and the ratio of carbon fiber was approximately 40%. The thickness of HFRP plates were set to 5.0, 7.0, 9.3 mm with the arrangement of the number of GF and CF ply. The low-viscosity matrix resin (Toray ACE AUP40) which is widely used to CF sheets bonding system for concrete structures, was selected in consideration to the workability in situ. Figure 2 shows the fabrication of the HFRP plate by VaRTM technology. The HFRP plates were fabricated in size of 700x700 mm at room temperature.

![Figure 2 Fabrication of HFRP plate by VaRTM technology](image)

**Table 1 Laminate configuration**

<table>
<thead>
<tr>
<th>Design thickness $t_f$ (mm)</th>
<th>Ratio of CF (%)</th>
<th>Equivalent elastic modulus (kN/mm²)</th>
<th>Laminate configuration</th>
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<td>7.0</td>
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<td>9.3</td>
<td>39.2</td>
<td>59.8</td>
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</tbody>
</table>

*Test procedure*

Tensile test was based on ACI 440 (ACI 2004,) and conducted using the universal testing machine of 1,000 kN loading capacity. The loading speed was set to 2.0 mm/min. Figure 3 shows the configuration of tensile test specimens. The both ends were reinforced by steel tubes and expanded resin.

![Figure 3 Tensile test specimen](image)

Compressive test was based on JIS K 7018 (equivalent to ISO/DIS 14126) and conducted using the universal testing machine of 100 kN loading capacity. The loading speed was set to 0.5 mm/min. Figure 4 shows the
configuration of compressive test specimens. The both ends were reinforced by bonded CFRP plates of 2 mm thickness.

Test result and discussion

Tables 2 and 3 show the properties of tensile and compressive tests, respectively. Table 2 shows tensile strength and tensile elastic modulus were approximately 1400 N/mm², 60 kN/mm² respectively, and were lower with increasing the thickness. It was found that the tensile elastic modulus was slightly higher than design equivalent elastic modulus as shown in Table 1. Although the coefficient of variation was larger with increasing the thickness, the value of C.V. was 8.5% in tensile strength and the value was 6.4% in tensile strength, respectively and these values were enough low. The values of poisson's ratio were the same value in spite of the thickness and were approximately 0.30.

Table 3 shows the values of compressive strength were larger with increasing the thickness and the value of compressive elastic modulus was slightly small in the case of 9.3 mm thickness, however the values were the same or more than the design value. It was found that the scatter in compressive properties was small so that the values of C.V. were smaller than that of tensile test. In addition, the values of poisson's ratio were the same value in spite of the thickness and were approximately 0.33.

<table>
<thead>
<tr>
<th>Thickness (mm)</th>
<th>Volume fraction $V_f$ (%)</th>
<th>Number of samples</th>
<th>Strength (N/mm²)</th>
<th>Elastic modulus (kN/mm²)</th>
<th>Poisson's ratio</th>
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<td>C.V.</td>
<td>Average</td>
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<td>7.0</td>
<td>5.7</td>
<td>55.2</td>
<td>5</td>
<td>438.2</td>
<td>0.066</td>
</tr>
<tr>
<td>9.3</td>
<td>8.5</td>
<td>50.8</td>
<td>5</td>
<td>451.8</td>
<td>0.016</td>
</tr>
</tbody>
</table>

It was found that the C.V. of strength and elastic modulus is the range between 2 and 8.5%. Therefore, the quality of HFRP plates fabricated by VaRTM technology was as equivalent to pultrusion profiles. Tables 2 and 3 also show the average values of thickness of specimen and the volume fraction of fiber $V_f$. These tables indicate that the measured thickness are thicker than the design thickness and $V_f$ is smaller with decreasing the thickness. It was confirmed that the values of $V_f$ are approximately 50% in spite of the thickness in this study.

Figures 5 and 6 show the failure modes in tensile and compressive test. In most specimens, the tensile failure occurred in the middle of specimen. The maximum tensile forces were larger with increasing the thickness, so that the pull out failure in steel grips occurred before tensile failure. Therefore, the valid data in the case of 9.3 mm thickness was four specimens. In compressive test, the failure occurred between the grips, and in many specimens, the failure occurred near the grips.
EVALUATION OF BONDED JOINTS BETWEEN STEEL PLATE AND HFRP PLATE

Fabrication of Test Specimens and Experimental Program

HFRP plates, which were the same fiber configuration as shown in Table 1, were fabricated and bonded on the steel plates by VaTRM technology. Figures 7 and 8 shows the fabrication of specimen by VaRTM technology and the configuration of test specimen. The cantilever flexure test was selected as a simple loading system and the bonding strength between steel plates and HFRP plates were evaluated in this system (Shimizu et al. 2014). Table 4 shows the material property of steel plate. The surfaces of steel plates (L850×B150×t12 mm) were treated by disk grainder and cleaned by acetone. Four spesimens were prepared in each case varying the thickness.

![Figure 7 Fabrication of specimen by VaRTM technology](image1)

![Figure 8 Configuration of Test specimen](image2)

Table 4 Material property of steel plate (JIS SM490Y)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield strength</td>
<td>427.3 N/mm²</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>549.6 N/mm²</td>
</tr>
<tr>
<td>Young’s modulus</td>
<td>211.0 kN/mm²</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.28</td>
</tr>
<tr>
<td>Elongation</td>
<td>38.7 %</td>
</tr>
</tbody>
</table>

Figure 9 shows the end of HFRP plate (7 mm thick) bonded by VaRTM. The end faces of HFRP plate were different shapes. The end shapes of 5 mm and 7 mm thick were smooth tapered shapes and that of 9.3 mm thick were almost vertical or rough shapes.

Moreover, in order to compare bonding strength by VaRTM, the four test specimens (No. 9.3e) were prepared as follows: the HFRP plate (L300×B100×t9.3mm) by VaRTM were bonded on the steel plates by the epoxy resin adhesive (Konishi E258RW). The method of surface treatment were sand blast in steel plates and sand paper in HFRP plates. The end face of HFRP plate was finished vertically as shown Figure 10.

![Figure 9 End of HFRP plate bonded by VaRTM](image3)

![Figure 10 End of HFRP plate bonded by epoxy resin](image4)

The cantilever flexure test was conducted using electro-hydraulic servo fatigue testing machine (load capacity: 100 kN). The concentrated load was applied by displacement control (0.05 mm/sec) until a clear change of strain in the end of the HFRP plate. Figure 11 shows the setup of the cantilever flexure test. Strain gages were installed on the upper and lower sides in the end and center of the HFRP plate in order to investigate the failure and the effect of strengthening. The location of strain gages (upper side: U1-U4, lower side: L1-L4) were also shown in Figure 8.
Effect of Strengthening by Externally Bonded HFRP Plates

Table 5 shows the thickness, the lowering rate of the strain in the center of steel plate (L4) and the debonding failure load. The theoretical values of the lowering rate were calculated using the average elastic modulus of HFRP plates in tensile and compressive tests and the young's modulus of the steel plate as shown in Tables 2 to 4, and were based on the equivalent bending rigidity as a perfect composite section and the bending stress of cantilever beam at the location of strain gage No. L4. The thickness in calculation were used the measured values. The thickness of HFRP plates bonded by VaRTM were slightly thicker than the thickness of coupon specimens as shown in Figures 2 and 3.

Table 5 shows that the experimental values of the reducing rate of steel strain are good agreement with the theoretical values and in. Moreover, the case of 9.3 mm thick results that the experimental value is slightly larger than the theoretical value, and shows a tendency to decrease the reducing rate. The case of specimen by epoxy resin adhesive also shows the same tendency and the equivalent effect of strengthening.

<table>
<thead>
<tr>
<th>Thickness (mm)</th>
<th>Number of specimens</th>
<th>Reducing rate of steel strain</th>
<th>Debonding failure load (kN)</th>
<th>C.V. (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design</td>
<td>Measured</td>
<td>Theoretical</td>
<td>Experimental</td>
<td>Average</td>
</tr>
<tr>
<td>5</td>
<td>4.9</td>
<td>5</td>
<td>0.77</td>
<td>0.75</td>
</tr>
<tr>
<td>7</td>
<td>6.0</td>
<td>5</td>
<td>0.68</td>
<td>0.67</td>
</tr>
<tr>
<td>9.3</td>
<td>9.2</td>
<td>5</td>
<td>0.48</td>
<td>0.53</td>
</tr>
<tr>
<td>9.3e</td>
<td>8.5</td>
<td>4</td>
<td>0.48</td>
<td>0.56</td>
</tr>
</tbody>
</table>

Debonding Failure Load

Figure 12 shows the relationship between the load and the longitudinal strain (U1 to U4) in case of 7-1. The steel strain of U1 is located at the part without strengthening. The strain of HFRP plate of U2, U3, U4 are located at 5, 15 and 150 mm from the end of HFRP plate in the fixed side. This figure shows all the strain straight increases with increasing load and in case of the strain of U2 to U4, the strain near the end of the HFRP plate is lower. In generally, the stain near the end of the HFRP plate was larger by bending moment based on the perfect composite section. Because the stress is not sufficiently transmitted due to the shear deformation of the HFRP plate.

On the other hand, it was found that the strain of U2 and U3 decreased drastically at the load of 1.72 kN. The minimum load, where the maximum strain occurred at the end of the HFRP plate, is specified as the debonding failure load, because it is considered that the strain decreases if the failure occurs at the end of the HFRP plate. Although, in the case of 7-1, the debonding failure load of U2 and U3 are the same value, the debonding failure load is evaluated using the strain of U2 in all the specimens, since the initiation of failure occurs at the end of the HFRP plate. In the case, where the failure load was not evaluated using the strain of U2, it was evaluated using the strain of U3.

Figure 13 shows the relationships between the applied load and the longitudinal strain in the case of 9.3 mm thick and Table 5 also shows the average value of the debonding failure load and the coefficient of variation in all the specimens. This figure indicates that the strain in the case of the epoxy resin adhesive is lower than that in the case of the VaRTM. This reason can be explained with the influence of the shear deformation depending on the thickness of adhesive layer.
Table 5 shows that the debonding failure load in VaRTM is the maximum value in 7 mm thick and the minimum value in 5 mm thick, and the debonding failure occurred under the bending yield load at the fixed end of the cantilever beam. In addition, the debonding failure load in the epoxy resin adhesive is lowest in all the specimens. This can be explained by the following reasons; in the fabrication process of VaRTM, the end face becomes the tapered shape and the stress concentration is reduced at the end of the HFRP plate. The coefficients of variation in debonding failure load are approximately 30% in VaRTM, and 20% in the epoxy resin adhesive. The debonding failure load varied widely in VaRTM. Because the end faces of the HFRP plates were not controlled in VaRTM process and were different shapes as mentioned above. It is possible to arrange an optional shape and easy to taper the end of HFRP plate by the VaRTM technology.

CONCLUSIONS

The results of tensile and compressive tests indicated that the coefficients of variation are small, so that the HFRP members fabricated by VaRTM technology have high quality. In addition, they have the high fiber volume of fraction and the effect of reinforcement. Moreover, the results of evaluation of bonded joints between the steel plate and the HFRP plate by the cantilever flexure test indicated the debonding failure load is generally high, although they varied widely. In our future work, this technology will be promoted for practical use. In ongoing project, the VaRTM technology has been developed for repair of steel plate girder ends with vertical stiffeners damaged by corrosion.

ACKNOWLEDGMENTS

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REFERENCES


FATIGUE LIFE EXTENSION OF A STEEL PLATE WITH AN EDGE CRACK BY CFRP PLATES BONDING

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ABSTRACT

Fatigue failure is one of several failures in steel structures. In recent years, the application of bonding of carbon fiber reinforced polymer, CFRP, to the fatigue damaged member has been investigated for the retrofitting of existing steel structures. To investigate the extended fatigue life of the single edge cracked steel plate strengthened with CFRP, fatigue tests, FE analysis and theoretical prediction of fatigue crack propagation were carried out in this study. The experimental results showed that reinforcement with CFRP was effective to reduce crack growth ratio and the analytical investigation with FE analysis showed the stress intensity factor (SIF) could be effectively reduced by bonding of CFRP. Furthermore, a SIF of an edge crack covered with CFRP plates was estimated by the theoretical equation based on the linear fracture mechanics and crack propagation analysis was conducted by using the SIF. As the results of the crack propagation analysis, the estimated fatigue life of an edge crack repaired by bonding CFRP plates tended to be smaller than the fatigue life given by the tests.

KEYWORDS

CFRP plate, edge crack, axial loading, stress intensity factor, crack propagation analysis.

INTRODUCTION

At the weld joints of steel structures, initiation of fatigue cracks has been widely reported. Bonding of carbon fiber reinforced polymer, CFRP, has been developed to recover the corroded steel member or repair of fatigue cracks in steel structures (e.g. National Research Council, 2007; Chacon et al., 2004; Phares et al, 2003; and, Roach et al, 2008).

The previous researchers have investigated the extended fatigue life of crack repaired by CFRP bonding by the experiments and FE analysis. Sean et al. (2003) revealed that bonding of CFRP is effective to prevent crack initiation by experiments with notched steel plate and FE analysis focusing on stress concentration factor. Ye Huawen et al. (2010) investigated fatigue performance on tension steel plates strengthened with prestressed CFRP laminates by simple fracture mechanics model based on measured stress. Wang, H. T. et al. (2013) revealed the effect of FRP configurations on the extended fatigue life by FE analysis focusing on SIFs. Colombi, P. (2015) investigated the fatigue crack growth in CFRP-strengthened steel plates with edge crack by experiment, and the crack propagation analysis based on the FE results of crack opening displacement were conducted to confirm the reduction of a SIF. However, in these researches, the reduction of SIFs with CFRP-strengthened crack were evaluated by experimental results or FE analysis.

On the other hand, some researchers have tried to give the SIFs with CFRP-strengthened crack in the simple equation based on the linear fracture mechanics and composites mechanics (Liu et al, 2009 and Matsumoto et al, 2013). However, these researches were focused on CFRP-strengthened center-crack, therefore, the equation based on the linear fracture mechanics and composites mechanics to evaluate the SIF of CFRP-strengthened edge-crack have not been proposed.

This research conducted the fatigue tests on CFRP-strengthened edge-crack to investigate the fatigue life extension by bonding of CFRP. Additionally, the simple equation based on the linear fracture mechanics and composites mechanics to evaluate the SIF of CFRP-strengthened edge-crack was proposed.
FATIGUE TESTS

Specimen Geometry and Loading

The dimensions of the specimen used a steel plate of JIS-SM490Y are shown in Figure 1. In the specimen, an edge crack of length 10mm is installed at the center of the steel plate. CFRP plates were applied to the both sides of the steel plate, as shown in Figure 1. Longitudinal elastic modulus of CFRP plates is 170kN/mm², elastic modulus of adhesive is 1358N/mm². During the fatigue tests, the stress range was controlled as 80 or 100N/mm² with the stress ratio of zero.

Fatigue test results

Figure 2 shows the relationship between the crack length and the number of cycles. The Specimens N and N+C are the control specimen without CFRP plate and the specimen with CFRP plates, respectively. In these figures, the results of crack propagation analysis mentioned in the latter chapter are also shown as the soiled black and red lines. Comparison of theoretical analysis results and test results is discussed in the latter Chapter. As seen in Fig.2, the existing fatigue lives, which fatigue crack propagated to 25mm, in the Specimen N+C are 50 times longer than those in the Specimens N. Furthermore, CFRP plates worked without debonding during the fatigue tests.

FINITE ELEMENT ANALYSIS

FE model

By using FE model, loading of the Specimens N and N+C shown in Fig.1 is simulated. Table 1 shows the material properties of steel, CFRP plates and adhesive. Material properties of CFRP plates were calculated based on the theory of composite material. Transverse elastic modulus of CFRP plates shown in Table 1 is smaller than that assuming CFRP plates as isotropic material \( G = E/(2(1+v)) \), since transverse elastic modulus of the matrix resin composing CFRP plates is small. The previous equation of SIF for a crack covered with CFRP plates does not consider the shear deformation of CFRP plates itself (Matsumoto et al, 2013). Therefore, for the comparison, FE analysis assuming CFRP plates as isotropic material, of which Elastic modulus is 170kN/mm² and Poisson’s ratio is 0.34, was conducted additionally. In FE analysis, uniform axial loads of 1N/mm² are subjected to the both edges of the model.
FE results of the SIF

Figure 3 shows the FE results of the SIFs. In this figure, the theoretical results of the SIFs as described in the next Chapter are also shown as the dot-line, solid and dashed lines. As can be seen in Fig.3, the SIF is significantly reduced by bonding CFRP plates in both case of anisotropic and isotropic materials. The FE results of isotropic material is smaller than those of anisotropic material. This is because the stress distribution of anisotropic CFRP plates, which consider the shear deformation of the matrix resin composing CFRP plates, at the right on the crack is smaller than that of the isotropic CFRP plates.

![Figure 3 FE results of SIFs](image)

### STRESS INTENSITY FACTOR OF A CRACK REPAIRED BY CFRP PLATE

#### Equation of a SIF of a crack repaired by CFRP plate

Figure 4 shows the superposition principle of the SIF of a crack repaired by CFRP plates. Assuming the steel plate subjected to tensile stress of $\sigma_0$ (State 0 in Fig.4), the nominal stress, $\sigma_n$, is reduced by bonded CFRP plate, since the CFRP bonded section can be regarded as the composite section (State 1 in Fig.4). On the other hand, since the CFRP plates carry the additional stress at the crack, which cannot be distributed by the crack, the crack opening displacement covered with CFRP plate is reduced compared with unreinforced state. It can be assumed that this additional stress distributed to the CFRP plate on the crack, $\sigma_n$, is applied to the crack surface to close the crack (State 1 in Fig.4). State 1 in Fig.4 can divide into States 2 and 3 based on the superposition principle. Thereby, the SIF of a crack repaired by bonded CFRP plates, $K_I^{(0)}$, can be calculated by using the SIF of State 2 in Fig.4, $K_I^{(2)}$, and the SIF of State 3 in Fig.4, $K_I^{(3)}$.

$$K_I^{(0)} \equiv K_I^{(1)} = K_I^{(2)} - K_I^{(3)} \quad (1)$$

By using Eq.(1), the SIF of a center cracked plate covered with CFRP plates, $K_I^{(0)}$, is given in the following equation, which takes into account the crack opening caused by the thickness of adhesive $h$ (Matsumoto et al, 2013).

$$K_I^{(0)} \equiv \sigma_n F_w \left(1 - F_g \frac{h}{a} \right) \quad (2)$$

where $a$ is the edge crack length, $F_w$ is the correction factor for a central crack in a plate of finite width and $F_g$ is the geometry correction factor caused by CFRP bonding.

Generally, in case of an edge crack, $F_g = 1.12$, which is the free surface correction factor, is multiplied by Eq.(2), since the crack opening displacement of an edge crack exposed to the free surface becomes larger than that with a central crack. On the other hand, in case of an edge cracked plate all of which is covered with CFRP plates, it can be assumed that the effect of the free surface does not need to be considered in the calculation of the SIF, since the crack opening displacement is suppressed by bonded CFRP plates. Therefore, the SIF of an edge crack covered with CFRP plates is calculated by Eq.(2).

### Table 1 Material properties

<table>
<thead>
<tr>
<th></th>
<th>$E$ (GPa)</th>
<th>$v$</th>
<th>$G$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>200</td>
<td>0.3</td>
<td></td>
</tr>
<tr>
<td>Adhesive</td>
<td>1.36</td>
<td>0.4</td>
<td></td>
</tr>
</tbody>
</table>

$E$: Elastic modulus (kN/mm$^2$), $v$: Poisson’s ratio, $G$: Transverse elastic modulus (kN/mm$^2$)
As mentioned in FE results of the SIFs, shear deformation of CFRP plates have modest effect on the SIF. Especially, in the poor fiber volume content of CFRP, the shear deformation becomes high. In the previous study, the conversion thickness of adhesive $h'$, which considered the shear deformation of CFRP plates caused by the matrix resin, was proposed by the following equation (Shirai et al, 2015).

$$h' = h + \frac{t_c}{3} \left( G_{czx}(1-V_c) + G_{e}V_c \right)$$  \hspace{1cm} (3)

where $h$ is the actual thickness of adhesive, $t_c$ is the thickness of CFRP plates, $G_e$ is the transverse elastic modulus of adhesive, $G_{czx}$ is the transverse elastic modulus of CFRP plates and $V_c$ is the fiber volume content.

Since $G_eV_c$ in Eq.(3) is negligibly small, $h'$ is able to assume by the following equation.

$$h' = h + \frac{t_c}{3} (1-V_c)$$  \hspace{1cm} (4)

In this paper, the conversion thickness $h'$ is used in Eq.(2) as a thickness of adhesive layer to calculate the SIF of an edge crack covered with CFRP plate. In this specimen, conversion thickness $h'$ is calculated as 0.37mm.

**Theoretical results of the SIF**

In Fig.3, the theoretical results of the SIFs are shown as dot-line, soiled and dashed lines. The theoretical results with adhesive thickness of $h = 0.25\text{mm}$ is good agreement with the FE results with isotropic CFRP plates, and the theoretical results with adhesive thickness of $h' = 0.37\text{mm}$, which included the shear deformation of CFRP plate, is also good agreement with the FE results with anisotropic CFRP plates.

**CRACK PROPAGATION ANALYSIS**

To clarify the fatigue life extension repaired by CFRP bonding, crack propagation analysis is conducted by using the following equation.

$$\frac{da}{dN} = C \left( \frac{\Delta K}{\Delta \sigma} \right)^m$$  \hspace{1cm} (5)

where $N$ is the number of cycles, $\Delta K_{th}$ is the threshold stress intensity factor of steel. In this paper, $\Delta K_{th}$ is assumed as 0. Material constants $C$ and $m$ are calculated by the relationship between $\frac{da}{dN}$ and $\Delta K$ given by the Specimen N ($C=5.781\times 10^{-17}$, $m=4.323$). Figure 5 shows the relationship between the SIF range divided by the stress range $\Delta K/\Delta \sigma$ and crack length $a$. The test results of $\Delta K/\Delta \sigma$ are calculated in the following equation.

$$\frac{\Delta K}{\Delta \sigma} = \left( \frac{\frac{da}{dN}}{C} \right)^{\frac{1}{m}}$$  \hspace{1cm} (6)

As can be seen in Fig. 5, the test results of $\Delta K/\Delta \sigma$ in the Specimen N+CFRP is smaller than the FE results of $\Delta K/\Delta \sigma$. One of the reasons is assumed that the adhesive thickness varies through the specimen and a part of adhesive thickness is thinner than the measured value. The results of the crack propagation analysis by using Eq.(5) was also compared with test results in Fig.2. In the Specimen N+CFRP, the analytical results of crack propagation velocity is higher than that of the test results, as seen from Fig.2. This is because the test results of $\Delta K/\Delta \sigma$ in the Specimen N+CFRP is smaller than the FE results of $\Delta K/\Delta \sigma$, as shown in Fig.5.
CONCLUSION

This paper presented the simple equation of Stress Intensity Factor of an edge cracked plate repaired by bonding CFRP plates, based on the linear fracture mechanics. By the proposed equation, it could successfully consider the effect of the shear deformation of CFRP plates on Stress Intensity Factor, and the estimated results of Stress Intensity Factor was good agreement with the FE results. However, the estimated fatigue life of an edge crack repaired by bonding CFRP plates tended to be smaller than the fatigue life given by the tests.

REFERENCES


FRCM for Strengthening and New Construction
ON TENSILE BEHAVIOR OF FRCM MATERIALS: AN OVERVIEW

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ABSTRACT

In the context of the strengthening for existing structure, the scientific community has recently focused its attention on fibre-reinforced cementitious matrix (FRCM) composites. Such materials are made from fabric meshes embedded in a cementitious matrix instead of an epoxy resin, such as FRP material. Unlike fiber-reinforced polymers (FRPs), characterized by an elastic tensile behaviour, the FRCMs exhibit an anelastic tensile behaviour with cracks formation. The FRCM plastic behaviour in tension made the tensile characterization more complex with respect to FRP composite and more parameters need to be evaluated.

For their widespread use, it is necessary to draft guidelines on their mechanical characterization, useful to designers, offering different types of tensile and shear-bond testing procedures. This paper will present an overview of research advancements on the mechanical behaviour of FRCM composites, summarizing the various types of experimental testing setup, identifying the critical issues and peculiarities. Moreover, a database, collecting the experimental results on tensile tests, will be presented and discussed focusing on the influence of single components properties on the global mechanical parameters of FRCM composite.

KEYWORDS

FRCM, tensile testing procedure, crack propagation.

INTRODUCTION

The advantages of FRCM material for retrofitting masonry structural elements are provided by the use of inorganic matrices with respect to epoxy resin, showing high fire resistance, permeability, applicability on wet surfaces, reversibility, easiness and reduced costs of installation.

The FRCM (Fabric-reinforced cementitious matrix) composite materials are composed by an inorganic matrix characterized by mortar or concrete in which a bidirectional large mesh textile reinforcement is embedded. The textile can be coated in order to ensure protection and to improve the adhesion between the matrix and the textile. The forefather of the FRCM is the ferrocement, a composite material made with a high quality mortar in which grid of steel bars of small diameters are inserted. More recent are the TRC (Textile Reinforced Concrete) and the TRM (Textile Reinforced Mortar) composites, which are used as strengthening material for masonry or concrete structure. The FRCM includes both the families.

The increasing adoption of FRCM materials as reinforcement of existing structures, leads to the need, for manufacturers and designers, to define some material characterization procedures and design rules. The scientific community is working in this direction, through various research projects, giving indications about the experimental evaluation of the mechanical behaviour of FRCMs. The qualification procedures of FRCM composites should reproduce the real behaviour of the material considering its different applications, especially when used as reinforcement of existing structures (flexural strengthening of reinforced concrete beams, reinforcement of arches and vaults, confining material of concrete columns, strengthening of masonry walls), evaluating both the constitutive tensile behaviour of the FRCM, and also the adhesion between it and the support material (masonry or concrete) and between mortar and fibres. In the present work the first issue is investigated.

The first experimental tests addressed to the mechanical characterization of FRCM specimens, that considered the overall material, and not the component materials (textile and matrix) individually, date back to the nineties (Dugas \textit{et al.} 1998). From those years many studies have been conducted by researchers at RWTH Aachen University School, Germany, (Jesse 2004, Molter \textit{et al.} 2004, Hegger \textit{et al.} 2006, Hartig \textit{et al.} 2009) who carried out and suggested different types of direct tensile tests, assessing advantages and disadvantages. Afterwards, more recently, a satisfying work was carried out by RILEM TC 232 and TC250.

Pointing towards the knowledge of the FRCM tensile constitutive law, the aim of this work is to gather, to compare and to discuss the results, available in literature, of experimental campaigns based on tensile tests on FRCM specimens. The comparisons are based on more than 350 data. Subdividing the experimental data in homogeneous
families based on test set-up typology and fibre grid, the average stress and strain values of the FRCMs tensile constitutive law are computed.

**TENSILE BEHAVIOUR: TESTING PROCEDURES**

The most common test typology used to characterize the tensile behaviour of FRCMs is the direct tensile test. The difficulty to define a tensile characterization procedure, able to determine the tensile constitutive behaviour of FRCMs, lies in the strong influence that various parameters, such as the material type, the specimen geometry, the specimen preparation, the test set-up, as well as the type of measuring instruments, have on the experimental results. Generally, the direct tensile tests, used for the mechanical characterization of different materials, consist in submitting a specimen (with rectangular or dogbone shape) under uniaxial tensile loading that increases monotonically. The specimen is clamped at the extremity ends by a stiff clamping jaws, that transmit and spread the tensile load to the specimen. It was noticed that for FRCM specimens the clamping typology strongly influences the tensile strength, the stresses distribution, the cracking evolution and the failure modes of the specimen (Hartig *et al.* 2011, De Santis *et al.* 2014, Carozzi *et al.* 2015, Arboleda *et al.* 2015). This is the main characteristic that differentiates the various types of tensile testing proposed by several authors.

![Figure 1](image.png) Direct tensile test type on FRCM specimen: rigid clamping (Type A), soft clamping (Type B) (Arboleda *et al.* 2015). In table the range values of the geometrical parameters are reported. \( L \) is the length, \( b \) the width, \( t \) the thickness of the specimen.

<table>
<thead>
<tr>
<th></th>
<th>( L ) [mm]</th>
<th>( b ) [mm]</th>
<th>( t ) [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type A</td>
<td>300-900</td>
<td>30-100</td>
<td>6-12</td>
</tr>
<tr>
<td>Type B</td>
<td>200-600</td>
<td>30-180</td>
<td>7-12</td>
</tr>
</tbody>
</table>

As it emerges from the collection of experimental data, there are two basic types of direct tensile tests. One type is known as "Set-up A" (or clevis grip or rigid load application), and the other as "Set-up B" (or clamping grip or soft clamping) (Hartig *et al.* 2010, Arboleda *et al.* 2015). The two types are distinguished by the mode of the tensile load transmission from the testing machine to the specimen through the gripping plates (Figure 1). The set-up A allows to distribute the tensile load on the specimen through adhesive forces and shear stresses. This test set-up consists in applying two bolted rigid plates, subjected to tensile loading, glued with epoxy resin on the sample extremities, avoiding the slipping between the steel plates and the specimens, from which the connotation of "rigid" load application system. This clamping system can be designed to allow rotations to ensure a uniaxial tensile stress state and prevent parasitic bending moments. The set-up B permits the stress transmission between the specimen and gripping plates through Coulomb friction, applying a pressure orthogonal to the plate faces. In this case, the slipping between the plates and the specimens may occur. This clamping type, is, in fact, knew as "soft" clamping. The advantage of the set-up A methods, according to the technical acceptance criteria AC434, lies in the better stress distribution in the specimen, which avoids the mortar cracking in the area close to the clamping zone. It well reproduces the stress state when the FRCM is used as strengthening material and the textile is not constrained at the extremities, because of the typical specimen failure mode due to the slippage of the grid. The set-up B method is advantageous for the easy execution. The typical failure mode of the FRCM specimen is ascribable to the tensile failure of the grid because of the clamping pressure that avoids the sliding of the textile. This method can be more acceptable when the tensile constitutive behaviour of the FRCM want to be known.

In order to evaluate the stress-strain curve is necessary to define the measuring length on the sample and the type of measuring instruments for the strains evaluation. The most common measuring setup consist in applying on the coupon LVDTs, extensometers or utilize the photogrammetry measurement, that is a refined method to evaluate the crack propagation in the specimen during the test. Comparisons between the curves resulting from different
measurement systems have proven that the obtained strains are very similar (Contamine et al. 2011, Hegger et al. 2006). As suggested in Colombo et al. 2013 and Contamine et al. 2011, it can be useful to apply two LVDTs on both the specimen sides, in order to evaluate potential flexural effects due to the loading eccentricity or to evaluate possible defects of asymmetry in the specimen. According to De Santis et al. 2014 the measurement system applied on the entire free length of the specimen are effective if there are not sliding in the gripping zone, and however the measurement instrument base has to be large with respect to the crack spacing.

The typical uniaxial tensile behavior of FRCM is characterized by a trilinear curve composed of three branches, each of which identifies a different stage of cracking. In Figure 2 a generic curve is shown, in which the stresses $\sigma$ are calculated as the ratio between the load $F$ and the area of the dry textile cross section $A_f$, in order to account the tension stiffening phenomenon, while the strains are estimated on a representative length of the sample through measuring instruments. The first stage of the curve reproduces the tensile behaviour of the not cracked specimen, with a stiffness $E_t$. The second stage describes the behaviour of the specimen during the cracking propagation in which the cracks develop gradually and the mortar is still able to contribute to the composite stiffness: the tension stiffening phenomenon occurs. The third branch is influenced by the grid tensile strength and not by the already cracked mortar, and it is characterized by the slope $E_f$ parallel to $E_t$.

![Figure 2 Tensile stress-strain law representative of the FRCM behaviour.](image)

**COLLECTED DATABASE ON FRCM TENSILE TEST**

In the present work the data regarding the mechanical and geometrical characteristic of FRCM specimens made with different materials, the test set-up and the results of more than 20 experimental campaigns (441 tensile tests), available in literature, were collected. In Table 1 the range values of the mechanical parameters for each FRCM family represented by the grid nature, and the relative data numerosity are listed. The large range of values for cementitious matrix strength and for grid properties used in the experimental tests make the understanding of tensile FRCM behaviour particularly complex.

The experimental results were analysed by comparing the tensile trilinear behaviour (Figure 3) characterized by the three stages, such as the cracking point ($\sigma_{cr}$, $\varepsilon_{cr}$ in Figure 2), the transition point ($\sigma_{tr}$, $\varepsilon_{tr}$ in Figure 2) and failure point ($\sigma_{fu}$, $\varepsilon_{fu}$ in Figure 2). Due to the different grids, the tested specimens are grouped according to the bidirectional textile reinforcement composed by AR-Glass, Aramid, Carbon, Basalt, PBO yarns or steel unidirectional fibres. In the following graphs the colours of the reported data represent the different materials of Table 1.

<table>
<thead>
<tr>
<th>Grid</th>
<th>AR Glass</th>
<th>AR Glass + Aramid</th>
<th>Carbon</th>
<th>PBO</th>
<th>Steel</th>
<th>Basalt</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_t$ [GPa]</td>
<td>21.3 - 64.8</td>
<td>101.5</td>
<td>190 - 263</td>
<td>215 - 277</td>
<td>184</td>
<td>67</td>
</tr>
<tr>
<td>$f_{fu}$ [MPa]</td>
<td>533 – 1010</td>
<td>1829.3</td>
<td>1300 - 4800</td>
<td>3397 - 5800</td>
<td>1500 - 3200</td>
<td>1160</td>
</tr>
<tr>
<td>$\varepsilon_{fu}$ [%]</td>
<td>-</td>
<td>2.15</td>
<td>1.18 - 1.6</td>
<td>-</td>
<td>2.2 – 2.24</td>
<td>1.82</td>
</tr>
<tr>
<td>Grid size (warp) [mm]</td>
<td>4.4 - 8.3</td>
<td>10 - 15</td>
<td>10 - 20</td>
<td>10</td>
<td>0.83 – 6.35</td>
<td>25</td>
</tr>
<tr>
<td>Grid size (weft) [mm]</td>
<td>5 - 10.2</td>
<td>18 - 20</td>
<td>20</td>
<td>10</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td>Matrix</td>
<td>$E_m$ [GPa]</td>
<td>16.79 - 46</td>
<td>10.3 - 22</td>
<td>7 - 31.5</td>
<td>&gt;6</td>
<td>11.4 – 31.5</td>
</tr>
<tr>
<td></td>
<td>$f_{um}$ [MPa]</td>
<td>1.74 - 9.17</td>
<td>4.4 – 10.3</td>
<td>3.3 - 5.5</td>
<td>2 - 4.75</td>
<td>2.5 – 5.5</td>
</tr>
<tr>
<td></td>
<td>$f_{cm}$ [MPa]</td>
<td>36.1 - 115</td>
<td>12.3 – 56.3</td>
<td>20 - 49</td>
<td>15 - 33.9</td>
<td>20.6 - 55</td>
</tr>
</tbody>
</table>
The stress that characterizes the cracking stage $\sigma_c$, is compared with the mortar tensile strength (Figures 4a, b). The stress is computed with respect to the specimen area, through the ratio between the experimental cracking load $F_c$ and the specimen area $A_\sigma$. The different sizes of the point reported in the charts state the data numerosity for the average evaluation: bigger point size represents the mean value of results obtained by more tests. In Set-up B the first crack occurs for a stress lower with respect to the mortar strength. For the scatter of experimental data a direct relation between stress and mortar strength seems not is showed, instead for Set-up A the experimental data practically stay on the bisectrix, that means, as expected, the cracking stress depends directly on mortar tensile strength.

In Figures 4 (c, d) the slope of the third branch of the FRCM tensile stress-strain law in average $E_3$, computed as reported in Figure 2, is represented versus the elastic Young’s modulus of the textile material $E_t$. It emerges that, effectively, for the data obtained with the set-up B (Figure 4d) the slope $E_3$ of ultimate stage of the curve, in which the specimen is significantly cracked and the mortar is not able anymore to contribute at the composite stiffness, is ruled by the grid tensile behaviour with stiffness $E_t$. Instead in Figure 4(c), the third branch stiffness is lower than that of the grid, this is probably due to the eventual textile slippage into the mortar for the case of the FRCM tested with the set-up A.

CONCLUSIONS

A collected database on FRCM tensile tests with 441 data was presented. Attention was posed on grouping the data in homogeneous families based on grid nature and on the adopted experimental set-up with the final goal to evaluate which experimental procedure correctly describes the actual tensile behaviour of FRCM for design purpose. A first analysis of parameters describing the tensile behaviour was performed and the dependency of FRCM mechanical properties on the single components properties was highlighted. In the future work more data.
will be collected with focus on the cracking propagation stage depending on cementitious matrix and on grid nature, with also the aim of establishing the tension-stiffening model of FRCM materials by accounting the effects of various parameters, such as the test setups, the types and the mechanical properties of reinforcing fibers as well as the grid sizes.

ACKNOWLEDGMENTS

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REFERENCES


INFLUENCE OF THE FIBER TYPE AND MATRIX AGE ON THE BONDING OF FRCM COMPOSITE STRIPS APPLIED TO CONCRETE SUBSTRATES

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ABSTRACT

Fiber reinforced cementitious matrix (FRCM) composites represent an alternative to fiber reinforced polymer (FRP) composites for strengthening existing civil structures. FRCM is comprised of fibers, usually in the form of a textile with an open-mesh configuration, embedded in an inorganic mortar matrix. It shares the advantages of FRP systems and overcomes some of its drawbacks, which makes it suitable for a wide range of applications. Although research on this topic is still scarce, it has been shown that debonding represents a key factor in its performance. In order to gain more insight in this issue, a series of classical push-pull single-lap direct shear tests were carried out on basalt, carbon, and glass FRCM-concrete joints. The results allow for comparing the performance of the joints with carbon, basalt, and glass fibers in terms of applied load – global slip response and failure mode. The influence of matrix age was also investigated. The curing time of the mortar matrix was found to influence the load carrying capacity of carbon FRCM – concrete joints.

KEYWORDS

FRCM, basalt, carbon, glass, concrete, debonding.

INTRODUCTION

Fiber reinforced polymer (FRP) composites have been proven to be effective in repairing or strengthening structures. The most common types of fibers used in FRP composites are carbon, glass, or more recently basalt. Although FRPs have shown good results in terms of bonding and application, they present some drawbacks. These include: regulations on how to handle the epoxy bonding agents due to the toxicity of the components, low permeability and diffusion tightness, and poor compatibility with a concrete substrate (Blanksvärd et al. 2009).

Fiber reinforced cementitious matrix (FRCM) composites are intended as an alternative to FRP composites for strengthening existing masonry and concrete structures. FRCM is comprised of fibers, usually in the form of a textile with an open-mesh configuration, embedded in an inorganic mortar matrix. It shares the advantages of FRP systems and overcomes the above-mentioned drawbacks.

Research on the use of FRCM composite in strengthening applications is still in its infancy, and comprehensive studies of the stress transfer mechanism and bond properties of FRCM composites are not yet available. Only a few studies have been carried out to compare the behavior of FRCM – concrete joints (D’Antino et al. 2015), and a systematic study on the influence of fiber type is not yet available. Moreover the bond between the base concrete and the matrix is not completely understood, especially since the bond between the base concrete and the mortar can be influenced by drying shrinkage (Orosz et al. 2010).

It is important to note that strengthening of a structure usually requires it to be temporarily taken out of service, so short curing times are desirable, which is an advantage of using FRP composites. Therefore, for the case of FRCM composites, it is of interest to understand the influence of matrix curing time on composite strength development.
EXPERIMENTAL PROGRAM

The composites were comprised of a bidirectional balanced fiber net made of basalt, carbon, or glass and a cementitious mortar matrix. Mechanical properties of the fabrics are provided in Table 1 in terms of ultimate tensile strength $f_t$, ultimate tensile strain $\varepsilon_f$, and modulus of elasticity $E_f$ as given by the manufacturer. Fiber net geometrical properties are also presented in Table 1. The cross sectional area of the bundles $A_b^*$ was provided by the manufacturer. The fiber nets are characterized by fiber bundle spacing $b$, width $b^*$, and thickness, $t^*$. Values of $b^*$ in Table 1 were measured, and values of $t^*$ were calculated as $A_b^*/b^*$.

Table 1 Properties of reinforcement textiles

<table>
<thead>
<tr>
<th>Textile type</th>
<th>Acronym</th>
<th>$b$ [mm]</th>
<th>$A_b^*$ [mm$^2$]</th>
<th>$b^*$ [mm]</th>
<th>$t^*$ [mm]</th>
<th>$f_t$ [MPa]</th>
<th>$\varepsilon_f$ [%]</th>
<th>$E_f$ [GPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basalt</td>
<td>B</td>
<td>25 x 25</td>
<td>1.45</td>
<td>2</td>
<td>0.725</td>
<td>3200</td>
<td>30</td>
<td>90</td>
</tr>
<tr>
<td>Carbon</td>
<td>C</td>
<td>20 x 20</td>
<td>0.94</td>
<td>3</td>
<td>0.313</td>
<td>4700</td>
<td>18</td>
<td>240</td>
</tr>
<tr>
<td>Glass</td>
<td>G</td>
<td>25 x 25</td>
<td>1.25</td>
<td>2</td>
<td>0.625</td>
<td>2000</td>
<td>30</td>
<td>70</td>
</tr>
</tbody>
</table>

Note 1. Value reported by the manufacturer

The matrix was a ready-mixed cementitious non-shrink, fiber reinforced compound mortar, and it was used in each composite. The matrix used in this study was not specifically designed for improved bonding with respect to any of the fiber nets studied. It is a commercially-available repair mortar for both concrete and masonry applications. The compressive strength $f_{cm}$ and flexural strength $f_{cf}$ of the mortar were determined at various ages according to the standards (EN 12390-3 2009) and (EN 12390-5 2009), respectively. The prisms sizes for the flexural tests were 40x40x160 mm. The two halves of each prism were placed between 40x40 mm loading plates of a press to determine the compressive strength of the mortar. The average value of $f_{cm}$ was 16.4 MPa, 25.0 MPa, 32.8 MPa, at 3, 7, and 28 days, respectively, and the average value of $f_{cf}$ was 3.63 MPa, 4.55 MPa, and 6.83 MPa, at 3, 7, and, 28 days, respectively. The average compressive strength of the concrete prisms was 59.3 MPa determined from six 150 mm cubes tested according to (EN 12390-3 2009) at 28 days.

The composite strips were bonded to the surface of unreinforced concrete prisms. The concrete prisms were 125x125x500 mm. The fiber strips were 500 mm long with three longitudinal fiber bundles. The width $b$ of the composite strip was chosen as a function of the spacing $b$ and width $b^*$ of the bundles in the fiber net, so that three bundles of the net would be fully embedded in the matrix. The composite width was 75 mm for basalt and glass FRRCM specimens and 60 mm for carbon FRRCM specimens. The bonded length used in this study was $l = 330$ mm and was chosen based on previous studies of PBO FRRCM-concrete joints (D’Ambri $et al.$ 2012, D’Antino $et al.$ 2014). The concrete surface was cleaned and prepared with light sandblasting prior to applying the composite. Wood forms were first attached to the concrete prism to allow for good control of the bonded area (Figure 1a), then a mortar layer of 4 mm thickness was cast onto the concrete surface. The fiber net was subsequently placed on the surface of the fresh mortar. A second set of forms was attached on top of the fiber net to secure it in place and to cast the external layer of mortar with a thickness of 4 mm (Figure 1b). The specimens were then cured under normal ambient conditions until the day of testing.

![Figure 1 a) Specimen preparation before applying the composite, b) test specimen after application of the composite, c) experimental setup](image-url)
A single-lap (direct) shear test setup was adopted in this study. A push-pull configuration was used in which the composite fibers were pulled while the concrete prism was restrained (Figure 1c). A similar test setup has been used in (Sneed et al. 2014), among other studies, to test other FRCM composite-concrete joints. Two aluminum plates were glued to the end of the fiber net using a thermosetting resin (Figure 1c). The aluminum plates were then introduced between two steel plates that were bolted together to clamp the fiber net. The steel clamp was attached to a pinned joint through which the load was applied to the bare fiber strips. The concrete prism was restrained using a 50 mm thick steel plate anchored with four 18 mm diameter rods to the fixed end of the testing machine. The loading was applied in displacement (machine stroke) control at a rate of 0.05 mm/min using a closed loop servo-hydraulic universal testing machine.

Global (loaded end) slip \( g \) was measured using two linear variable transducers (LVDTs) that were attached to the concrete surface close to the composite side edges and oriented parallel to the bonded length. The rod of the LVDTs reacted against an aluminum L-profile that was glued to the bare fiber net outside of the bonded area. The term \( g \) is defined as the displacement between the fibers just outside of the composite at the loaded end and the adjacent surface of the concrete prism (Carloni et al. 2014), and is computed as the average of the two LVDT measurements.

Twenty-seven specimens were tested in this study, and the test variables were fiber type (carbon, glass, or basalt) and matrix age (3, 7, or 28 days). Test specimens are listed in Table 2. Each series is identified by acronyms indicating the textile type and age in days of the composite at test date. The identification of a particular specimen within a series is indicated by the digit at the end of the group name (example: B_03_2 indicates basalt fibers, age at test date of 3 days, second specimen of the series). Results of basalt FRCM specimens tested at 3 days were disregarded and are not included due to problems with the data acquisition system. Specimens with glass fibers were tested only at 7 and 28 days.

Table 2 Test results

<table>
<thead>
<tr>
<th>Series</th>
<th>Specimen</th>
<th>( P_{\text{max}} ) [kN]</th>
<th>( \varepsilon_{\text{pmax}} ) [%]</th>
<th>( \sigma_{\text{pmax}} ) [MPa]</th>
<th>( g_{\text{pmax}} ) [mm]</th>
<th>( P_{\text{cr}} ) [kN]</th>
<th>( g_{\text{cr}} ) [mm]</th>
<th>Failure mode</th>
<th>Criterion</th>
</tr>
</thead>
<tbody>
<tr>
<td>C_03_</td>
<td>1</td>
<td>0.932</td>
<td>1.377</td>
<td>330</td>
<td>0.245</td>
<td>-</td>
<td>-</td>
<td>FP</td>
<td>Pass</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.908</td>
<td>1.342</td>
<td>322</td>
<td>1.185</td>
<td>-</td>
<td>-</td>
<td>FP</td>
<td>Pass</td>
</tr>
<tr>
<td>C_07_</td>
<td>1</td>
<td>1.217</td>
<td>1.798</td>
<td>432</td>
<td>1.22</td>
<td>-</td>
<td>-</td>
<td>FP</td>
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</tr>
<tr>
<td></td>
<td>2</td>
<td>1.374</td>
<td>2.030</td>
<td>487</td>
<td>1.194</td>
<td>-</td>
<td>-</td>
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<td>Pass</td>
</tr>
<tr>
<td></td>
<td>3</td>
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<td>-</td>
<td>-</td>
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<td>Fail</td>
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<td>1.151</td>
<td>276</td>
<td>0.738</td>
<td>-</td>
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<td>1.634</td>
<td>2.414</td>
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<td>-</td>
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<tr>
<td></td>
<td>3</td>
<td>1.947</td>
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<td>B_07_</td>
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<td>1.423</td>
<td>0.912</td>
<td>0.458</td>
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<tr>
<td></td>
<td>2</td>
<td>1.891</td>
<td>4.830</td>
<td>435</td>
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<td>1.387</td>
<td>1.495</td>
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<td></td>
<td>3</td>
<td>2.2</td>
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<td></td>
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<td>888</td>
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<td></td>
<td>6</td>
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<td>7.466</td>
<td>672</td>
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<td>1.529</td>
<td>6.796</td>
<td>408</td>
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<td>3</td>
<td>1.499</td>
<td>6.662</td>
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<td>1.696</td>
<td>-</td>
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</tr>
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<td>G_28_</td>
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<td></td>
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<td>8.713</td>
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<td>0.152</td>
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<td>-</td>
<td>D</td>
<td>Fail</td>
</tr>
</tbody>
</table>

D – delamination of composite from concrete substrate
DC – delamination within the composite (between the two matrix layers)
FP – fiber pull-out (debonding of fibers from the embedding matrix)
RESULTS AND DISCUSSION

Summary of results

Table 2 reports the peak load $P_{\text{max}}$, global slip at peak load $g_{\text{Pmax}}$, the equivalent stress in the fiber bundles at peak load $\sigma_{\text{Pmax}}$ computed as $P_{\text{max}}/n\Lambda_b^*$ where $n$ is the number of longitudinal fiber bundles ($n=3$), the equivalent maximum strain in the fibers $\varepsilon_{\text{Pmax}}$ computed as $\sigma_{\text{Pmax}}/E_f$, the load at first crack formation in the composite $P_{\text{cr}}$, and the corresponding global slip $g_{\text{Pcr}}$, and observed failure mode.

In Table 2 the symbol “-” indicates that no data were recorded for the particular parameter. The presence of “-” across the entire row indicates that the composite strip detached from the concrete prism prematurely, usually during clamping of the aluminum plates in the steel clamp, especially for basalt and glass specimens since the fabric stiffness of the two fiber nets is much higher compared to that of the carbon net. For a successfully tested specimen, the presence of “-” in $P_{\text{cr}}$ and $g_{\text{Pcr}}$ indicates that no cracks were noticed on the surface of the composite before the peak load was reached. Some specimens showed an unreasonably low peak load compared to other specimens within the same group. The authors believe that this could be caused by the nonuniform distribution of the applied load among the longitudinal fiber bundles. It was also observed that specimens with a lower peak load showed a high rotation of the L-profile. In (Carloni et al. 2014) a criterion is used to determine whether the results of a specimen can be considered reliable or not based on the rotation of the rigid plate used to measure $g$. In this study specimens that showed “large” rotations of the L-profile were marked as “Fail” in Table 2 and were disregarded from further analysis. Large rotations were determined by comparing the measured load-displacement responses of the individual LVDTs. It should be noted that in the case of the present study this check was only qualitative since detailed bond-slip relationships for the tested composites are not currently available. Usually, large rotations were also characterized by lower peak loads.

(a) Carbon FRCM
(b) Basalt FRCM
(c) Glass FRCM

Figure 2 Applied load $P$ - global slip $g$ response of FRCM-concrete joints
Applied load $P$ - global slip $g$ response

Figure 2 shows the applied load $P$ – global slip $g$ of specimens in the carbon, basalt, and glass FRCM series, respectively. Only results of specimens with “Pass” in Table 2 are plotted. In each graph of Figure 2, plots with same line type correspond to specimens of the same series and tested at the same age. All specimens with carbon fibers (Figure 2a) failed by debonding of the fiber bundles from the embedding matrix. This represents a typical behavior observed for FRCM-concrete joints where the FRCM composite consists of uncoated, high modulus fibers such as carbon or PBO (D’Antino et al. 2014, Sneed et al. 2014). After an initial almost linear branch, the response becomes nonlinear as the microcracking occurs at the fiber-matrix interface. The applied load increases until the effective bond length is fully established and the bond mechanism reaches the free end of the bonded length (Carloni et al. 2014). After the peak load, the fiber bundles were observed to pull out of the matrix, and the load gradually decreases until it reaches constant plateau. The constant load after the bundles are completely debonded is associated with friction (interlocking) (D’Antino et al. 2014).

The load response of the basalt FRCM-concrete joints (Figure 2b) was characterized by an initial linear response, followed by a sudden decrease in load associated with an increase in global slip. These jumps were associated with the formation of cracks parallel to the composite width, and the delamination of the composite from the loaded end up to the location of the crack. Subsequently, the load increased and additional cracks were observed up to the peak load when the specimens failed by complete delamination of the composite from the concrete substrate. No damage was observed on the concrete surface, indicating that failure occurred at the matrix – substrate interface.

The limited number of specimens with glass fiber nets (Figure 2c) that were considered to “pass” the criterion showed a behavior similar to that of the carbon fiber specimens. The specimens show an initial linear behavior followed by a nonlinear branch as a result of fiber-matrix interface damage. The applied load increases up to the peak load where either fiber pull-out or complete composite delamination from the substrate was observed to occur. No damage was observed on the concrete surface after the composite detached.

Influence of fiber type

In this section a comparison of specimens tested at 28 days is carried out between the three series. Figure 3 shows the equivalent stress $\sigma$ - global slip $g$ relationship for all specimens that “passed” the criterion in Table 2. Plots with the same line type in Figure 3 correspond to specimens of the same series. The highest maximum stress was observed in specimens with basalt fibers (average of 837 MPa) at a corresponding slip of approximately 3 mm. The stress determined for specimens with carbon fibers (average of 635 MPa) was lower than that of the basalt but higher than that of the glass fibers (average of 523 MPa). The maximum stress in the carbon fibers was reached at a slip of approximately 1 mm. Delamination of the composite from the substrate was the dominant failure mode observed for basalt, whereas fiber pull-out was observed for carbon and glass. This suggests that the bond strength of the fiber-matrix interface for the basalt FRCM composite is higher than for carbon and glass FRCM composites. Since failure occurs at the weakest interface, for basalt it is the composite-substrate interface, therefore delamination occurred. It should be noted that the results are only valid for the materials used in this study.

Figure 3 Stress $\sigma$ - global slip $g$ response of carbon, basalt, and glass FRCM-concrete joints at 28 days
Influence of matrix age

Influence of curing time on the bond behavior on FRCM – concrete joints was studied for carbon, basalt, and glass FRCM composites. For the sake of brevity as well as limited available information, the discussion of the influence of matrix age is limited to specimens with carbon nets presented in Figure 2a. A clear difference in terms of peak load can be observed between tests carried out at 3, 7, and 28 days. The average maximum load at 3 days was 0.92 kN (51% of the 28 day average maximum load), at 7 days was 1.29 kN (72% of the 28 day average maximum load), and at 28 days was 1.79 kN. On the other hand, the residual capacity did not appear to be significantly influenced by matrix age, which may be explained by the fact that the residual capacity is associated with mechanical interlocking and not bond (D’Antino et al. 2014). A similar increase in maximum load was also observed for basalt and glass FRCM, however, more tests need to be carried out for a better understanding.

CONCLUSIONS

In this paper, experimental results of 27 single-lap direct shear test of carbon, basalt, and glass FRCM composites bonded to a concrete substrate are presented, of which 16 were found to show reliable results. The load responses and failure modes differed between the tested composites. Carbon and glass FRCM specimens failed by fiber debonding from the embedding matrix, while basalt FRCM specimens failed by delamination from the concrete substrate. Basalt and glass FRCM specimens showed cracks on the matrix surface during testing. The load at which the first crack occurred was generally much lower than the maximum load. In contrast carbon FRCM specimens did not show any cracks on the matrix surface.

Based on limited test results, despite having a lower capacity compared to basalt, carbon FRCM could be more suitable for strengthening application because of the increased rigidity and lack of matrix cracking. More tests should be carried out in order the better identify parameters that influence the load response of FRCM composites using commercially available fiber nets and textiles.

Results from this study show that matrix age influences the strength of carbon FRCM composites, especially the bond strength of the fiber-matrix interface.

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REFERENCES

ABSTRACT
Shear failure of reinforced concrete (RC) beams is an undesirable mode of failure due to its sudden and brittle nature and thus needs to be carefully evaluated when planning a strengthening intervention. The use of fiber reinforced polymer (FRP) composites has shown to be capable of providing an adequate increase in shear strength. However, in recent years, there is interest in developing new techniques in which the positive attributes of FRP are utilized but some of its drawbacks are overcome. Among these techniques, fiber reinforced cementitious matrix (FRCM) composites, in which the organic resins are replaced by inorganic mortars, have shown promising results. In this paper, a bibliographical review of the available literature on FRCM shear strengthening of RC beams is carried out. Two available design models are evaluated using a database compiled by the authors. The review shows that FRCM is able to provide an increase in strength and performance comparable to RC beams strengthened with FRP. However, the models are not able to accurately predict the behavior of FRCM strengthened beams.

KEYWORDS
FRCM, FRP, reinforced concrete, shear, strengthening.

INTRODUCTION
Fiber reinforced polymer (FRP) composites have proved their efficiency for upgrading existing structures for bending, shear, and confinement, allowing a time/cost efficient intervention. However, some limitations, mainly related to the use of organic resins, have been pointed out (Al-Salloum et al., 2012). Fiber reinforced cementitious matrix (FRCM), a composite material that employs an inorganic cement-based matrix instead of an organic matrix, allows for overcoming some of the drawbacks presented by FRP composites. While research carried out on the topic is still scarce, the effectiveness of this technique for flexural and shear strengthening and confinement of axially/eccentrically loaded RC elements has been confirmed (ACI 549.4R, 2013; Triantafillou and Papanicolaou, 2006; Ombres, 2015).

This paper summarizes the major findings on the topic of shear strengthening of RC beams with FRCM composites. In the first part, a detailed bibliographical review of the available literature on the shear strengthening of RC beams using FRCM composites is carried out. In the second part, two published design models proposed to predict the contribution of the FRCM composite to the shear strength of RC beams, including the ACI 549.4R (2013) expressions, are assessed using a database of experimental results compiled by the authors.

EXPERIMENTAL DATABASE
A total of 14 articles related to shear strengthening of RC beams using FRCM composites were found in the technical literature. Table 1 summarizes the references including the author(s), year, beam cross-sectional shape, number of strengthened beams tested, type of failure observed, strengthening configuration, and presence of anchors for each article. Eighty-three strengthened beams are included in the database.
Table 1 Summary of references on shear strengthening of RC beams using externally bonded FRCM composites

<table>
<thead>
<tr>
<th>REFERENCE</th>
<th>YEAR</th>
<th>BEAM SHAPE</th>
<th>NUMBER OF STRENGTHENED BEAMS</th>
<th>FAILURE MODE</th>
<th>STRENGTHENING CONFIGURATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Triantafillou, T. and Papanicolaou, C.</td>
<td>2006</td>
<td>R</td>
<td>3</td>
<td>FLEXURE</td>
<td>SHEAR</td>
</tr>
<tr>
<td>Bruckner, A. et al.</td>
<td>2006</td>
<td>R</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bruckner, A. et al.</td>
<td>2008</td>
<td>T</td>
<td>9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Blanksvard, T. et al.</td>
<td>2009</td>
<td>R</td>
<td>7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Al-Salloum, Y. et al.</td>
<td>2012</td>
<td>R</td>
<td>8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Contamine, R. et al.</td>
<td>2013</td>
<td>R</td>
<td>6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Azam, R. and Soudki, K.</td>
<td>2014</td>
<td>R</td>
<td>6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Baggio, D. et al.</td>
<td>2014</td>
<td>R</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tzoura, E. and Triantafillou, T.</td>
<td>2014</td>
<td>T</td>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Escrig, C. et al.</td>
<td>2015</td>
<td>R</td>
<td>6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jung, K. et al.</td>
<td>2015</td>
<td>R</td>
<td>8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ombres, L.</td>
<td>2015</td>
<td>R</td>
<td>7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tetta, Z. et al.</td>
<td>2015</td>
<td>R</td>
<td>8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Trapko, T. et al.</td>
<td>2015</td>
<td>R</td>
<td>1</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**TOTAL**: 83    7    76    25    50    (13)    8

*R=Rectangular, T=T-beam
*Yielding of longitudinal steel bars followed by concrete crushing
*Failure mode related to the FRCM debonding, fibres rupture, diagonal tension, and/or yielding of the stirrups.
*SB=Side bonded, U=U-Wrapped, W=Fully wrapped.
*Numbers in parentheses indicate tests that include anchors

Evaluation of the Database

In this paper, the shear strength provided by the FRCM system \( V_{FRCM} \) is calculated by subtracting the shear strength of the corresponding control beam \( V_{CON} \) for each test. The ranges of the main geometrical and mechanical properties of the strengthened specimens and the FRCM composites are reported in Table 2 (a/d=shear span to effective depth ratio; \( f'_{c} \)=mean cylinder compressive strength of concrete; \( \rho_{long} \)=longitudinal steel reinforcement ratio, \( A_b/b_d \); \( \rho_w \)=internal transverse steel reinforcement ratio, \( A_w/b_w \); \( \rho_f \)=fiber reinforcement ratio, \( 2n t_f/b_w s_f \); \( \rho_{cm} \)=FRCM reinforcement ratio, \( 2(n+1)t_m/b_w s_f \); \( \varepsilon_{fu} \)=bare fiber ultimate strain; and \( f'_{c,cm} \)=cementitious compressive matrix strength; where \( A_b \)=longitudinal steel reinforcement area; \( A_w \)=internal transverse steel reinforcement area; \( s \)=internal transverse steel reinforcement spacing; \( t_f \)=nominal thickness of fiber sheets; \( n \)=number of layers of fibers \( w_f \)=width of FRCM strips; \( s_f \)=spacing of FRCM strips; and \( t_m \)=nominal thickness of a matrix layer):

Table 2 Ranges of selected properties for FRCM strengthened RC beams

<table>
<thead>
<tr>
<th>PROPERTY</th>
<th>RANGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>( a/d )</td>
<td>2.22 - 4.90</td>
</tr>
<tr>
<td>( f'_{c} ), MPa</td>
<td>10.1 - 46.2</td>
</tr>
<tr>
<td>( \rho_{long} )</td>
<td>0.00792 - 0.05133</td>
</tr>
<tr>
<td>( \rho_w )</td>
<td></td>
</tr>
<tr>
<td>Number of fiber layers</td>
<td></td>
</tr>
<tr>
<td>( \rho_f )</td>
<td>0.000237 - 0.005588</td>
</tr>
<tr>
<td>( \rho_{cm} )</td>
<td>0.02667 - 0.2333</td>
</tr>
<tr>
<td>Fiber type</td>
<td>Basalt</td>
</tr>
<tr>
<td>( \varepsilon_{fu} )</td>
<td>0.007653 - 0.0315</td>
</tr>
<tr>
<td>( f'_{c,cm} ), MPa</td>
<td>21.8 - 86.7</td>
</tr>
</tbody>
</table>

463
Figure 1 (left) presents the variation of $V_{FRCM}/V_{CON}$ as a function of the average shear strength (stress) $v_{CON}$ of the control beam computed according to Eq. 1:

$$v_{CON} = \frac{V_{CON}}{b_{w}d}$$ (1)

In Figure 1 (left), it is observed that the increase in shear strength attributed to the FRCM composite varies from 3% to 195% with an average value of 54%. It also can be seen that the effectiveness of the system appears to decrease for larger values of $v_{CON}$.

Regarding failure mode, Triantafillou and Papanicolaou (2006) and Bruckner et al. (2006) witnessed fiber rupture and beam cracking clearly visible on the surface of the FRCM jacket for fully wrapped elements. Although detachment, which is described as debonding of the FRCM jacket from the substrate in this paper, is reported in some the references (Contamine et al., 2013; Ombres, 2015; and Jung et al., 2015), other failure modes such as diagonal tension, slippage of the fibers through the mortar, and partial fiber rupture, referred to herein as no-detachment failure mode, have also been described in the available literature (Al-Salloum et al., 2012; Escrig et al., 2015; Baggio et al., 2014; Azam and Soudki, 2014; and Tetta et al., 2015).

The possible interaction between the internal and external shear reinforcement for FRCM systems has been reported by Blanksvard et al. (2009) and Ombres (2015), who witnessed a significant reduction in the strain values measured in the stirrups of the strengthened beams when compared with the control beam. In Figure 1 (right), it can be seen that the effectiveness of the strengthening system reduces when the ratio of the axial stiffness of the stirrup to the fibers $\rho_{w}E_{s}/\rho_{f}E_{f}$ (where $E_{s}=\text{elastic modulus of stirrups}$, and $E_{f}=\text{elastic modulus of the bare fibers}$) increases.

The use of FRCM anchorages has shown mixed results. Baggio et al. (2014) evaluated the efficiency of FRP spike anchors for rectangular beams strengthened in shear with U-wrapped FRCM composites and reported that the beam with anchors showed an increase of only 3% over the strengthened beam without anchors. L-shaped steel sections were used by Bruckner et al. (2008) to anchor the FRCM system for U-wrapped T-beams. The increase in shear capacity varied from 19% for elements without anchors to a maximum of 29% when anchors were present, depending on the number of fiber layers. Tzoura and Triantafillou (2015) used a 3 mm thick curved steel section fixed to the slab with threaded rods to anchor FRCM U-wrapped T-beams. The increase in strength range from approximately 18% for elements without anchors to a maximum of 187% when anchors were present, depending on the density of the textile.

ASSESSMENT OF AVAILABLE MODELS

This section assesses two models proposed in the literature to estimate $V_{FRCM}$; the models by Triantafillou and Papanicolaou (2006) and ACI 549.4R (2013) are referred to herein as Model 1 and Model 2, respectively. Both models are based on the well-known truss analogy and differ mainly in the expression used to evaluate the stress (strain) in the FRCM system along the shear crack. Model 1 is based on the properties of the bare fibers, and its evaluation is carried out using a database (DB1) from the tests in Table 1 that excludes elements that included anchors and/or that failed in flexure. Model 2 is based on the mechanical properties of the FRCM system as a composite, which were reported only by four of the references (Ombres 2015; Contamine et al. 2015; Escrig et al. 2015; Jung et al. 2015). A subset of DB1 that includes tests from these four references, referred to as DB2, was used to evaluate the performance of Model 2. For each model, values of test-to-predicted ratios of $V_{FRCM}$, termed $V_{test}/V_{pred}$, are computed, where $V_{test}$ is the value of $V_{FRCM}$ determined from the experimental test by subtracting...
the shear strength of the corresponding control beam, and \( V_{\text{pred}} \) is the value of \( V_{\text{FRCM}} \) determined by the model. Average (AVG) and coefficient of variation with respect to the mean value (COV) values of \( V_{\text{test}}/V_{\text{pred}} \) are also reported for each model.

**Model 1: Triantafillou and Papanicolaou (2006)**

Model 1 was first presented for fully wrapped rectangular beams and then extended for U-wrapped elements (Tzoura and Triantafillou, 2015). The predicted additional shear strength provided by the FRCM system is computed by Eq. 2:

\[
V_{\text{FRCM}} = \rho_f \sigma_{\text{eff}} b_w d_f; \sigma_{\text{eff}} = 0.5E_f \varepsilon_{fu}
\]  

where \( d_f \) is the effective depth of the jacket. The effective stress in the FRCM system (\( \sigma_{\text{eff}} \)) is computed based on the average strain reached across the shear crack, taken as 50% of the ultimate strain of the bare fibers, \( \varepsilon_{fu} \). In Figure 2, values of \( V_{\text{test}}/V_{\text{pred}} \) are plotted versus \( V_{\text{test}} \).

For elements that failed by detachment of the strengthening system, Figure 2 (center) shows that Model 1 tends to overestimate the contribution of the FRCM composites, which is corroborated by an AVG=0.51. For the remaining elements, Figure 2 (right) shows that \( V_{\text{test}}/V_{\text{pred}} \) ratios seem to be directly proportional to \( V_{\text{test}} \). In order to calculate the effective average strain in the fibers (\( \varepsilon_{\text{eff}} \)), Eq. 2 is rearranged in the form of Eq. 3:

\[
\varepsilon_{\text{eff}} = \frac{V_{\text{test}}}{E_f b_w d_f}
\]

Figure 3, in which the values of \( \varepsilon_{\text{eff}}/\varepsilon_{fu} \) are plotted in terms of the \( \rho_f E_f/\Gamma_{c}^{2/3} \), shows that the ratio \( \varepsilon_{\text{eff}}/\varepsilon_{fu} \) tends to decrease with increasing \( \rho_f E_f/\Gamma_{c}^{2/3} \), with lower values of \( \rho_f E_f/\Gamma_{c}^{2/3} \) for elements that did not show detachment. Assuming a constant concrete strength of the substrate, this finding indicates that a less stiff strengthening solution, i.e. lower values of \( \rho_f E_f \), might avoid the onset of detachment.

**Model 2: ACI 549.4R (2013)**

\( V_{\text{FRCM}} \) of RC beams strengthened by continuous U-wrapped or continuous fully wrapped FRCM composite is computed according to Eq. 4:

\[
V_{\text{FRCM}} = n A_f \sigma_{\text{eff}} d_f; \sigma_{\text{eff}} = E_{\text{FRCM}} \varepsilon_{\text{eff}} = \varepsilon_{\text{FRCM,u}} \leq 0.004
\]  

In Eq. 4, \( A_f \) is the area of mesh reinforcement per unit width effective in shear, and \( n \) is the number of fiber layers. Figure 4 shows values of \( V_{\text{test}}/V_{\text{pred}} \) versus \( V_{\text{test}} \). For elements that failed by detachment of the strengthening system,
Figure 4 (center) shows that although most of the tests (9 out 13) fall below the $V_{\text{test}}/V_{\text{pred}}=1.0$ line, the value of AVG equal is 1.03. On the other hand, for elements that did not show detachment, Figure 4 (right) shows that values of $V_{\text{test}}/V_{\text{pred}}$ are considerably larger than 1.0. Thus the accuracy of Model 2 is highly affected by the failure mode considered.

![Graphs showing $V_{\text{test}}$ versus $V_{\text{pred}}$ for Model 2 with different failure modes](image)

In Figure 5 values of predicted (model) and calculated (Eq. 5) strain ratios $\epsilon_{\text{eff}}/\epsilon_{fu}$ are plotted against $\rho_{E_{\text{FRCM}}}f_{c}^{2/3}$. It can be seen that values of calculated $\epsilon_{\text{eff}}/\epsilon_{fu}$ for the model are always lower than 25%. The agreement between the calculated and predicted strain ratios appears to be clearer for elements that failed due to detachment. It is also interesting to note that beams that failed by detachment have values of $\rho_{E_{\text{FRCM}}}f_{c}^{2/3}$ larger than 0.003. This implies that less stiff solutions, either in terms of $E_{f}$ or $E_{\text{FRCM}}$, should be used in order to avoid detachment of the system from the substrate.

**Comparison of the performance for Models 1 and 2**

Table 3 summarizes the AVG and COV values determined for the 2 models studied. The best AVG value is observed from the results of Model 2 when elements with detachment are considered (1.03), but the largest AVG is also found for the same model (3.70) for elements that did not show detachment. Model 1 tends to highly overestimate the contribution of the FRCM system for elements that did not show detachment but presents a more consistent behavior in terms of COV for both failure modes.

<table>
<thead>
<tr>
<th>FAILURE MODE</th>
<th>MODEL</th>
<th>DB #</th>
<th>AVG</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Detachment</td>
<td>1</td>
<td>1</td>
<td>27</td>
<td>0.51</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>2</td>
<td>13</td>
<td>0.26</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>1</td>
<td>36</td>
<td>1.12</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>6</td>
<td>0.72</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>2</td>
<td>6</td>
<td>3.70</td>
</tr>
<tr>
<td>No detachment</td>
<td>1</td>
<td>1</td>
<td>63</td>
<td>0.86</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>19</td>
<td>0.40</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>2</td>
<td>19</td>
<td>1.87</td>
</tr>
</tbody>
</table>

\[
\epsilon_{\text{eff}} = \frac{V_{\text{test}}}{nA_{f}E_{\text{FRCM}}f_{f}}
\]
CONCLUSIONS

The experimental evidence gathered in this article proves that the FRCM system is able to provide an increase in the shear strength of RC beams from 3% to 195% with an average of 54%. A possible interaction between the internal transverse steel reinforcement and the FRCM system has been observed. This effect appears to be more pronounced for higher values of $\rho_s E_s / \rho_f E_f$. For fully wrapped elements, the failure mode has been associated with fracture of the fibers. For side bonded and U-wrapped beams, detachment of the FRCM jackets has been reported. However, failure without detachment has also been witnessed together with diagonal cracking, slippage of the vertical fibers through the mortar, and/or partial fiber rupture. According to the available experimental results, strengthening solutions with lower values of $\rho_s E_s / f'c^{2/3}$ or $\rho_f E_f / f'c^{2/3}$ might avoid the onset of detachment. Although Model 1 overestimates the additional shear strength provided by the FRCM composite, it presents a more consistent behavior regarding values of COV when compared to Model 2.

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STRENGTHENING OF SHEAR DEFICIENT REINFORCED CONCRETE BEAMS WITH TEXTILE REINFORCED MORTAR (TRM)

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ABSTRACT

One of the most commonly used technique for retrofitting and/or strengthening of reinforced concrete members is external bonding of fibre reinforced polymer (FRP) materials. Use of epoxies and resins to bind or impregnate the fibres has been seen as drawbacks in the FRP strengthening technique especially in the regions where high temperatures persist. Textile reinforced mortars (TRM), which uses cement-based inorganic mortars in place of organic binders, have emerged as a possible replacement of FRP to overcome the aforementioned drawbacks. This study focuses on studying the behaviour of shear-deficient RC beams strengthened with TRM and to compare the ultimate failure loads and modes of failure with control and RC beams detailed as per ACI 318-08. Overall eight beams, two control, two shear-deficient, and four strengthened, were tested under four-point bending with varying shear spans. Control beams were detailed in flexure and shear as per ACI 318-08, while shear deficient and strengthened beams have same flexural reinforcement as in control beams with no shear reinforcement. Two of the four strengthened beams were strengthened in shear with TRM by providing U-shaped TRM wraps in the shear span. Evenly distributed strips were provided in the shear span for one set of two beams while full wraps were provided in shear span in the remaining beams. Results indicate that use of TRM resulted in better performance of shear deficient RC beams and has a potential to be used in strengthening of RC beams.

KEYWORDS

Strengthening, shear-deficient, RC beams, textile reinforced mortar.

INTRODUCTION

Researchers in recent decades have focused their research on the use of fibre composites in reinforced concrete (RC) structures that needs strengthening and/or retrofitting due to deterioration resulting from number of causes. Fibre Reinforced Polymers (FRPs) have been frequently used in this regard both for research purposes and in strength restoration and/or upgrading of existing structures due to various reasons like upgrade of design codes, deterioration in structures or their members due to unforeseen loads, under design, change of usage modes etc. Despite of successful use of FRP in terms of strength upgradation (Teng et al. 2002), there are a few drawbacks in using FRP which have been identified by researchers including costly application, reduced fire resistance, potentially harmful solvents for workers involved in the installation of FRPs, poor applicability of FRP on moist surface and at colder temperatures (Aldea et al. 2007, Olivito et al. 2013). For this reason, researchers have looked into using cementitious binding materials which may replace the epoxies used with FRPs (Garcia et al. 2010) and help overcome the above mentioned problems. Preliminary researches related to fibre meshes embedded in cementitious mortar dates back to 1980’s (Gardiner et al. 1983). The interaction between fibre and matrix is replaced by grids of fibre, which are held together by knitting the fibre roving in at least two directions, which are commonly placed orthogonally. The spacing between the roving can be varied while manufacturing to obtain the desired mechanical properties of the textile grids and the distribution of the cementitious mortar through the openings. The cementitious material should possess high workability along with adequate shear and tensile strength to restrain early de-bonding. The combination of these fibre grids and cementitious matrix is commonly termed as Textile Reinforced Mortar (TRM) (Triantafillou et al. 2006), although various authors have referred to it with different names (Ombres 2011). The research being conducted in the area signifies that TRM can play a pivotal role for strengthening of structural members in terms of flexure, shear and confinement (Brückner et al. 2006, Triantafillou et. al 2013, Loreto et. al 2015).
This study focuses on studying the behaviour of shear-deficient RC beams, with varying shear spans, strengthened with TRM and to compare the ultimate load carrying capacities and modes of failure with control and RC beams detailed as per ACI 318-08.

**EXPERIMENTAL PROGRAM**

A total of eight beams were used in the study. All the beams were of 1.828 m length with a cross section of 152 mm × 203 mm. The longitudinal reinforcement was taken as 0.5 $\rho_{\text{max}}$ as per ACI 318-08 code. The study was conducted for the shear span-to-depth (a/d) ratios of 2.46 and 3.38. The control beams for both a/d ratios did not have any transverse reinforcement in form of shear stirrups. Two beams, one each for the a/d ratios, were cast having the same flexural reinforcement as control beams along with shear reinforcement as per ACI 318-08 code. Results of these beams were compared to the beams which had the same longitudinal reinforcement and no shear reinforcement. Remaining four beams were strengthened in shear for both the a/d ratios with U-shaped TRM wraps using two different schemes, one being fully wrapped in the shear span while the other one having 76 mm wide TRM strips at 125 mm centre to centre spacing. Reinforcement and strengthening details of the beams are shown in Figures 1-3. Nomenclature of the beams used in this research is given in Table 1.

<table>
<thead>
<tr>
<th>S. No.</th>
<th>Beam Description</th>
<th>a/d ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>CB-1, Control Beam with no stirrups</td>
<td>2.46</td>
</tr>
<tr>
<td>2</td>
<td>CB-2, Control Beam with no stirrups</td>
<td>3.38</td>
</tr>
<tr>
<td>3</td>
<td>ACI-1, Beam with shear reinforcement according to ACI Code</td>
<td>2.46</td>
</tr>
<tr>
<td>4</td>
<td>ACI-2, Beam with shear reinforcement according to ACI Code</td>
<td>3.38</td>
</tr>
<tr>
<td>5</td>
<td>TS-1, Beam with externally applied U-shaped TRM strips to resist shear</td>
<td>2.46</td>
</tr>
<tr>
<td>6</td>
<td>TS-2, Beam with externally applied U-shaped TRM strips to resist shear</td>
<td>3.38</td>
</tr>
<tr>
<td>7</td>
<td>TF-1, Beam with externally applied U-shaped TRM fully wrapped to resist shear</td>
<td>2.46</td>
</tr>
<tr>
<td>8</td>
<td>TF-2, Beam with externally applied U-shaped TRM fully wrapped to resist shear</td>
<td>3.38</td>
</tr>
</tbody>
</table>

Figure 1 R/F details for ACI beams with a/d ratio 2.46 and 3.38

Figure 2 R/F details for TS with a/d ratio 2.46 and 3.38
Concrete used for the beams was of 20.68 MPa (3 ksi) while yield strength of steel was found to be 414 MPa (60 ksi). For application of TRM, high strength mortar was used, with a 28 days strength of 27.6 MPa (4 ksi), and strips of 76 mm width were provided in U-shape. Centre to centre distance between them was 127 mm. The beams were tested in four point bending at a loading rate of 0.5 kN/min. Deflections and crack progression was noted up to the failure of the beams.

RESULTS AND DISCUSSIONS

Load Carrying Capacities and Failure Modes

Table 2 shows the failure modes and load carrying capacities of the control and strengthened RC beams. The control beams CB-1 and CB-2 failed in shear as expected. CB-1 failed before the start of major flexural cracking, while beam CB-2 failed at the yielding load with the development of shear cracks in the shear span along with major flexural cracking in the constant moment zone. Beams ACI-1 and ACI-2 failed in flexure as expected. The beam ACI-1 showed 18% increment in load carrying capacity as compared to CB-1 while beam ACI-2 carried almost the same load as CB-2 showing a marginal increase of 1%. Excessive flexural cracking was observed in the flexural region at failure in beams ACI-1 and ACI-2 showing the effectiveness of shear reinforcement provided in the beams.

Beams strengthened with TRM strips, TS-1 and TS-2, failed in flexure, with TS-1 beam showing 35% increase in load carrying capacity and TS-2 showing 9% increase in failure load in comparison with the respective control beams. Both the beams carried a higher load than the respective RC beams ACI-1 and ACI-2, 15% and 8% respectively. Failure in beams fully wrapped with TRM, TF-1 and TF-2, varied from post yield mortar failure to flexure. Beam TF-1 failed by failure of mortar immediately after yielding of reinforcement at 12% increased failure load as compared to CB-1. Beam failed almost at the same load as CB-2 and ACI-2 showing no increase in load carrying capacity. Excessive flexural cracking was observed in the flexural region at failure in beams TS-1, TS-2 and TF-2 while no major flexural cracking was observed in beam TF-1 due to the premature failure of mortar which can be attributed to improper bonding of TRM with concrete.

<table>
<thead>
<tr>
<th>S. No.</th>
<th>Beam</th>
<th>a/d ratio</th>
<th>Max. Load at yielding (kN)</th>
<th>Failure Load</th>
<th>Increase in load Carrying Capacity (%)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>CB-1</td>
<td>2.46</td>
<td>-</td>
<td>98</td>
<td>-</td>
<td>Shear</td>
</tr>
<tr>
<td>2</td>
<td>ACI-1</td>
<td>2.46</td>
<td>107</td>
<td>115.5</td>
<td>18</td>
<td>Flexure</td>
</tr>
<tr>
<td>3</td>
<td>TS-1</td>
<td>2.46</td>
<td>112</td>
<td>132.5</td>
<td>35</td>
<td>Flexure</td>
</tr>
<tr>
<td>4</td>
<td>TF-1</td>
<td>2.46</td>
<td>108</td>
<td>109.6</td>
<td>12</td>
<td>Post yield Mortar Failure</td>
</tr>
<tr>
<td>5</td>
<td>CB-2</td>
<td>3.38</td>
<td>70.2</td>
<td>72.6</td>
<td>-</td>
<td>Shear</td>
</tr>
<tr>
<td>6</td>
<td>ACI-2</td>
<td>3.38</td>
<td>70</td>
<td>73.5</td>
<td>1.2</td>
<td>Flexure</td>
</tr>
<tr>
<td>7</td>
<td>TS-2</td>
<td>3.38</td>
<td>70</td>
<td>79.1</td>
<td>9.0</td>
<td>Flexure</td>
</tr>
<tr>
<td>8</td>
<td>TF-2</td>
<td>3.38</td>
<td>68</td>
<td>70.2</td>
<td>0</td>
<td>Flexure</td>
</tr>
</tbody>
</table>

Load-deflection Curves

Load-deflection curves of control and strengthened beams with a/d ratio of 2.46 are shown in Figure 4. Load-deflection curve of control beam shows a sudden failure of beam with no clear yield point due to the development
of major shear crack in the shear span. Beam ACI-1, detailed with shear reinforcement steel according to ACI, showed a desirable flexural behaviour with very clear yield point and excessive flexural cracking as discussed in the preceding section. The beam TS-1, which was strengthened with TRM strips showed deformations comparable to that of ACI-1 but carried a higher load than ACI-1 as discussed earlier. At the ultimate load there is a sudden drop in the curve which can be attributed to the failure of mortar but before that excessive flexural cracking had occurred. Beam TF-1, fully wrapped with TRM in shear span, showed a distinct yield point but a sudden drop in load carrying capacity can be observed which is indicative of the post yielding shear failure due to premature failure of mortar used for external bonding of TRM. Yield load of beams ACI-1, TS-1 and TF-1 is almost similar indicating the effectiveness of internal and external shear reinforcement.

Figure 4 Load-Deflection Curves for a/d ratio 2.46

Figure 5 shows the load-deflection curves for control and strengthened beams with a/d ratio of 3.38. The control beam CB-2 showed a similar pattern as was observed in CB-1 i.e., sudden drop in load carting capacity due to major shear crack in shear span. Load-deflection curves of beams ACI-2, TS-2 and TF-2 are almost similar and represent typical flexure behaviour, showing a very distinct yield point and a ductile post yield response accompanied by excessive flexural cracking in pure flexural zone. A sudden drop is noticed in the case of TS-2 but that is after large deflections and excessive cracking making the failure ductile.

Figure 5 Load-Deflection Curves for a/d ratio 3.38

CONCLUSIONS

Following are the conclusions drawn from the study:

Use of TRM is effective in strengthening of shear deficient RC beams in shear as it contributes to shear capacity of the beams along with increasing the ductility and transforming the failure mode from brittle to shear as can be seen in the case of TS-1, TS-2 and TF-2.
For a/d ratio of 2.46, use of TRM resulted in maximum increase of 35% and 15% in load carrying capacity as compared to beams CB-1 and ACI-1.

For a/d ratio of 3.38, use of TRM resulted in maximum increase of 9% and 8% in load carrying capacity as compared to beams CB-2 and ACI-2.

TRM can be as effective in increasing shear capacity as internal shear reinforcement and has a potential to be used in strengthening of existing shear deficient RC beams.

ACKNOWLEDGMENTS

The authors are indebted to the Department of Civil Engineering at NED University of Engineering & Technology, Karachi, Pakistan and the University itself, in the pursuit of this work. Authors also gratefully acknowledge the assistance and support provided by the Muhammad Daniyal Awan, Muhammad Mutahir, Usama Naeem, Syed Muhammad Adeem, Hamid Kamal Pasha, Abdul Wasae Zaman.

REFERENCES

ABSTRACT

Lots of underground reinforced concrete (RC) structures, which were designed and constructed by former design standards, were collapsed or damaged due to uneven settlement in the Great East Japan Earthquake in 2011. It was pointed that the reinforced amount of the structures designed by the former standard was insufficient. A new reinforced method by using Carbon Fiber Reinforced Plastics (CFRP) grid with Polymer Cement Mortar (PCM) shotcrete for RC beams has been proposed. The objective of this study is to reveal the bonding behaviour of CFRP grid and evaluate the effect of shearing capacity of RC beams by using CFRP grid with PCM shotcrete method. It is carried out by two series of tests, such as bonding tests of CFRP grid and loading test of RC beams.

KEYWORDS

CFRP grid, PCM shotcrete method, RC member, bonding behaviour, strengthening effect.
grid as parameters in 5 types of specimens (Table 1), including 3 specimens for each type. The schematic of specimens (ex. FP50VH/FP50VL) and the installed position of strain gauges are shown in Figure 1. The results of bonding tests are summarized in Table 2 with mean values among the three specimens in each type shown. It can be seen that for all specimens, CFRP grids, which are not embedded in PCM (Base metal), were broken in the end. Relationship between grid points and the strain of CFRP grid at 12kN, 2/3 of ultimate load of the grid, is shown in Figure 2.

Table 1 Types of specimens (Bonding test)

<table>
<thead>
<tr>
<th>Type</th>
<th>Number of grid points</th>
<th>Interval of horizontal grid (mm)</th>
<th>Existence of vertical grid</th>
<th>Type of PCM</th>
</tr>
</thead>
<tbody>
<tr>
<td>FP50VH</td>
<td>5</td>
<td>50</td>
<td>Yes</td>
<td>High Strength</td>
</tr>
<tr>
<td>FP50VL</td>
<td></td>
<td>75</td>
<td>Yes</td>
<td>Low Elasticity</td>
</tr>
<tr>
<td>FP75VH</td>
<td>4</td>
<td>100</td>
<td>No</td>
<td>High Strength</td>
</tr>
</tbody>
</table>

Figure 1 Schematic of specimens (ex. FP50VH/FP50VL)

Table 2 Result of bonding test (Data Source: K. Sugiyama et al. 2011)

<table>
<thead>
<tr>
<th>Type</th>
<th>Maximum load (kN)</th>
<th>Maximum strain of CFRP grid (×10⁶)</th>
<th>Fracture Behavior</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Design</td>
<td>Experiment</td>
<td>Design</td>
</tr>
<tr>
<td>FP50VH</td>
<td>18.0</td>
<td>21.9</td>
<td>14000</td>
</tr>
<tr>
<td>FP50VL</td>
<td>17.7</td>
<td>19.1</td>
<td>11991</td>
</tr>
<tr>
<td>FP75VH</td>
<td>17.0</td>
<td>17.0</td>
<td>12025</td>
</tr>
<tr>
<td>FP100VH</td>
<td>15.6</td>
<td>15.6</td>
<td></td>
</tr>
<tr>
<td>FP100N</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 2 Distribution of strain of CFRP grid (P=12kN)

As a result, it can be seen that the strain of CFRP grid decreases as the number of grid point increases. It can be concluded that the tensile force acting on the CFRP grid is transmitted to the CFRP grid and concrete through
PCM. Compared with FP50VH and FP50VL (type of PCM as a parameter), the strain of No.5 grid point in FP50VH decreases to zero at the end, but that of FP50VL remains at about 2000×10⁻⁶. It is concluded that transmission ability of high strength PCM is better than that of low elasticity ones. The calculation method of the distribution ratio for each grid point is shown in Eq. 1 and Eq. 2.

\[ S = \sum_{i=1}^{4} \frac{A_i}{A_{400}} \times 100 \]  
\[ A_i = \frac{1}{2} \times (D_{i-1} + D_i) \times h_i \]  

where \( S \) is distribution ratio (%), \( h_i \) is interval of CFRP grid (mm), and \( D_i \) is strain of measurement point.

The results at 12kN, 2/3 of ultimate load of the grid, are shown in Figure 3. As the result, the distribution ratios of No.1 grid point in FP50VH (44%) and FP75VH (43%) are almost the same, while the distribution ration in FP100VH (56%) is higher than those of FP50VH and FP75VH. For specimens with vertical grid, it can be concluded that the distribution ratio of grid point becomes larger as the interval of horizontal grid increases. In addition, the total of distribution ratio is 70-80%, when the number of grid points is 2. And the total of distribution ratio of grid is over 90%, when the number of grid points is 3 or more. As a result, bonding behavior is sufficient.

**SHEAR STRENGTHENING EFFECT ON RC BEAM**

In this study, it is intended for sluice which is an important structure (Figure 4) placed across a river embankment to be used to drain waste water from protected inland and pump water from river. In the present method, the evaluation of shear strengthening on RC beams reinforced by FRP is based on the concept of effective strain of FRP rod (JSCE, 1996) or shear reinforced efficiency of FRP sheet (JSCE, 2000). However, evaluation of shear strengthening on RC beam reinforced by CFRP grid has not been established. In order to evaluate shear strengthening on RC beams by using CFRP grid with PCM shotcrete, loading tests of RC beams were carried out.

![Figure 4 Sluice](image)

Figure 5 shows production process of the specimens. Firstly, existing concrete is produced in I form. Span length of specimen is 4750mm and dimension of cross section is 750mm×500mm (height × width). After surface treatment by blasting, CFRP grid is attached to the surface of existing concrete by using rivet anchors as temporary fix. Then, primer is sprayed in order to prevent dehydration before PCM shotcrete.

Table 3 shows material properties for CFRP grid. Table 4 shows the types of specimens. There are 4 types of specimens including P-1 reinforced by CR4 CFRP grid, P-2 reinforced by CR8 CFRP grid, P-3 reinforced by CR4 CFRP grid whose interval of horizontal reinforcement (50mm) is 1/3 to other specimens (150mm), and P-4
reinforced by CR4&6 CFRP grid whose horizontal grid is CR6 and vertical grid is CR4. For all specimens, shear rebar at the SD295 D10 setting with a 250mm interval and main rebar at the SD345 D35 were used, where SD295 and SD345 are specified by JIS.

Figure 6 shows the shape of specimens and the situation of CFRP grid. In order to investigate the shear strengthening effect on RC beams reinforced by CFRP grid, strain gauges were placed at several locations of CFRP grid in both vertical and horizontal direction.

Table 3 Material properties for CFRP grid

<table>
<thead>
<tr>
<th>Type</th>
<th>Cross section area, mm²</th>
<th>Tensile strength, f₀, MPa</th>
<th>Yield strength, fₚ₀, MPa</th>
<th>Modulus of elasticity, E₀, Gpa</th>
</tr>
</thead>
<tbody>
<tr>
<td>CR4</td>
<td>6.6</td>
<td>1400</td>
<td>-</td>
<td>100</td>
</tr>
<tr>
<td>CR6</td>
<td>17.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CR8</td>
<td>26.4</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4 Types of specimens (Loading test)

<table>
<thead>
<tr>
<th>Type</th>
<th>Existing Portion</th>
<th>Reinforced Portion</th>
<th>CFRP grid (Interval: Vertical &gt; Horizontal)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P-1</td>
<td>D10@250</td>
<td>D35×10</td>
<td>CR4 (150×50)</td>
</tr>
<tr>
<td>P-2</td>
<td>D10@250</td>
<td></td>
<td>CR8 (150×50)</td>
</tr>
<tr>
<td>P-3</td>
<td>D10@250</td>
<td></td>
<td>CR4 (50×50)</td>
</tr>
<tr>
<td>P-4</td>
<td>D10@250</td>
<td></td>
<td>CR4&amp;6[1] (150×50)</td>
</tr>
</tbody>
</table>

[1]: Horizontal grid is CR6, vertical grid is CR4.

Figure 7 shows the relationship between load and displacement. All specimens were shear fractured in the end. Compared with P-1, which is reinforced by CR4, maximum load of P-2 reinforced by CR8 is improved by about
7%. And compared with P-1 and P-3, since the interval of horizontal grid of P-3 is 1/3 to that of P-1, maximum load of P-3 is improved by about 8%, and stiffness of curve is higher after about 600kN. At about 200kN, flexural cracks occurred on the surface of specimens. However, since the peeling of CFRP grid occurred early during loading test in P-4, the maximum load of P-4 is lower than that of P-1.

Figure 8 shows the relationship between load and strain of CFRP grid. It was seen that the obliquely shear cracks were confirmed in the surface of specimen at about 400kN during the loading test. The strain of vertical and horizontal grids both increase at the same time. It is found that both vertical and horizontal grids contribute to shear resistance. In Figure 8(a) (c), the strain of vertical grids abruptly become small, it is concluded that peeling occurs between CFRP grid and existing concrete before maximum load.

![Figure 7 Load-deflection curve of loading tests of RC beams](image)

![Figure 8 Load-Strain of shear rebar and CFRP grid](image)
Figure 9 shows crack distribution of RC beam at maximum load. During loading test of RC beam, the small width flexural cracks occur on the surface of specimen first. Then the shear cracks occur on the line connecting the load position and fulcrum. The length and width of cracks increase accordingly as the load increases. Finally, part of the upper edge of concrete is crushed. Furthermore, fracture situations of CFRP grid after loading test are confirmed by peeling the PCM, and the result is shown in Figure 9. It can be seen that parts of CFRP grid, which is above the shear cracks, are delaminated.

![Figure 9 Beam cracking](image1)

**P-1**

![Figure 9 Beam cracking](image2)

**P-2**

![Figure 9 Beam cracking](image3)

**P-3**

![Figure 9 Beam cracking](image4)

**P-4**

**Figure 9 Beam cracking (crack distribution at failure and fracture situations of CFRP grid)**

**EVALUATION OF SHEAR CAPACITY**

Shear Capacity is calculated by Eq. 3. $V_{con}$, $V_{pcm}$, $V_{se}$ and $V_{g}$ are share of shear capacity of concrete, PCM, shear rebar and CFRP grid. The calculation method of share of shear capacity of CFRP grid is shown in Eq. 4. The comparison of shear capacity is shown in Table 5. Calculated Value A is calculated by Eq. 4 where $\varepsilon_u$ is maximum tensile strain of CFRP grid (14000×10^{-6}). $\varepsilon_u$ of Calculated Value B is calculated by Eq. 5 which applied the concept of effective strain of CFRP rod (JSCE, 1996).

<table>
<thead>
<tr>
<th>TYPE</th>
<th>Experimental Value</th>
<th>Calculated Value A</th>
<th>Calculated Value B</th>
</tr>
</thead>
<tbody>
<tr>
<td>P-1</td>
<td>881</td>
<td>697 (1.26)</td>
<td>618 (1.43)</td>
</tr>
<tr>
<td>P-2</td>
<td>943</td>
<td>1163 (0.81)</td>
<td>707 (1.33)</td>
</tr>
<tr>
<td>P-3</td>
<td>950</td>
<td>993 (0.96)</td>
<td>667 (1.42)</td>
</tr>
<tr>
<td>P-4</td>
<td>860</td>
<td>712 (1.21)</td>
<td>632 (1.36)</td>
</tr>
</tbody>
</table>

[2]: Number in () shows Experimental Value / Calculated Value.
where $\varepsilon_s$ is Strain of CFRP grid, $E_w$ is Young’s Modulus of CFRP grid (N/mm²), $A_w$ is total area of cross section of CFRP grid in interval $S_t$ (mm²), $S_t$ is arrangement interval of CFRP grid (mm), $z = d/1.15$, and $\gamma_b = 1.15$.

$$
\varepsilon_s = \left(\frac{\rho_{str} E_{st}}{\rho_{web} E_w} \times 10^{-4}\right)
$$

where $f'_{cd}=(h/0.3)^{0.1}$, $p_{str}=A_r/(b_w d)$, $p_{web}=A_d/(b_s S_t)$, $f'_{cd}$ is design compressive strength of concrete (N/mm²), $d$ is effective height (m), $b_w$ is width of cross section of specimen (mm), $A_r$ is area of cross section of main rebar (mm²), and $E_{st}$ is Young’s Modulus of tensile rebar (N/mm²).

As a result, the ratio of Calculated Value A and its Experimental Value is higher than 1.00 in P-1 and P-4. However, in P-2 and P-3, the ratio of Calculated Value A and its Experimental Value is below 1.00. It can be seen that Calculated Value A is unable to evaluate the shear capacity. The ratio of Calculated Value B and Experimental Value is considerably higher than 1.00 in all specimens. It can be seen that Calculated Value B, which applies the concept of effective strain of CFRP rod, is excessive evaluation. In consideration of these observations, the improved design formula of shear capacity based on the concept of effective strain of CFRP grid should be proposed in order to make a better evaluation of shear capacity on RC beams by using CFRP grid.

Figure 10 shows the strain of CFRP grid in both vertical (as shown by □) and horizontal (as shown by ○) direction. In addition, results of a previous study (R. Guo et al. 2015) are also shown in Figure 10. A new calculation method of effective strain of CFRP grid (Eq. 6 shows) is proposed by regression analysis of the peeling strain of vertical grid, when peeling occurs between CFRP grid and existing concrete. Furthermore, it is found that horizontal grid also contributes to shear resistance. Therefore, Figure 11 shows the comparison between the strain of vertical and horizontal grid. It can be observed that relationship between the strain of horizontal grid and peeling strain of grid, when peeling occurs between CFRP grid and existing concrete. Furthermore, it is found that horizontal grid of effective strain of CFRP grid (Eq. 6 shows) is.

$$
V = V_{com} + V_{p Cosmic} + V_{str} + V_g
$$

$$
V_g = \frac{A_w \cdot E_w \cdot \varepsilon_s \cdot (\sin \alpha_w + \cos \alpha_w) \cdot z}{S_t \cdot \gamma_b}
$$

$$
\varepsilon_s = \left(\frac{\rho_{str} E_{st}}{\rho_{web} E_w} \times 10^{-4}\right)
$$

where $f'_{cd}=(h/0.3)^{0.1}$, $p_{str}=A_r/(b_w d)$, $p_{web}=A_d/(b_s S_t)$, $f'_{cd}$ is design compressive strength of concrete (N/mm²), $d$ is effective height (m), $b_w$ is width of cross section of specimen (mm), $A_r$ is area of cross section of main rebar (mm²), and $E_{st}$ is Young’s Modulus of tensile rebar (N/mm²).

$$
\varepsilon_{ver} = \left[\frac{1}{0.0352 \rho_{ver} + 0.0079}\right] \times 10^{-6}
$$

where $\rho_{ver}$ is area of cross section of vertical CFRP grid per unit length (mm²/mm)

$$
\varepsilon_{hor} = \alpha \times \varepsilon_{ver} = \alpha \times \left[\frac{1}{0.0352 \rho_{ver} + 0.0079}\right] \times 10^{-6}
$$

where $\alpha$ is comparison coefficient ($\alpha = 0.23$)

$$
V_g = \frac{2 \cdot E_{st} \cdot \rho_{str} \cdot \varepsilon_{ver} \cdot (\sin \alpha_w + \cos \alpha_w) + \rho_{web} \cdot \varepsilon_{hor} \cdot (\sin \alpha_w + \cos \alpha_w)}{\gamma_b \cdot \varepsilon_{ver}}
$$

where $\rho_{hor}$ is area of cross section of horizontal CFRP grid per unit length (mm²/mm)
CONCLUSIONS

The main findings of this investigation can be summarized as follows:

1. In bonding test, the strain of grid points reduces in turn, and tensile force to CFRP grid is transmitted by each grid point. The total of distribution ratio of grid is over 90% when the number of grid points is 3 or more. Bonding behaviour is sufficient.

2. From experiment data of the loading test of RC beams, it is concluded that part of CFRP grid peels off before maximum load. It is found that both vertical and horizontal grids contribute to shear resistance.

3. Calculation methods of effective strain of CFRP grid are proposed. The evaluation method by considering both vertical and horizontal grid is the most reasonable approach.

ACKNOWLEDGMENTS

The authors wish to express their gratitude and sincere appreciation to FRP Grid Method Association.

REFERENCES


ABSTRACT

A new strengthening composite and bonding system, namely Fibre Reinforced Polymer (FRP) grid-Ultra High Toughness Cementitious Composite (UHTCC), for Reinforced Concrete (RC) beams is explored in this paper. In this system, UHTCC is applied as the reinforcing material and the bonding agent between the FRP grid and the concrete substrate. Four RC beams externally strengthened with a composite layer of FRP-UHTCC applied to the side faces in the bending-shear regions for a four-point bending test setup and one similar test for a reference RC beam without strengthening were tested to investigate their structural performance. The reinforcement ratio of the FRP grid and the construction method of the UHTCC were the two test parameters. The experimental results highlighted two failure modes: 1) the typical shear-compression failure of concrete at the critical sections in the reference beam, and 2) the partial debonding failure of FRP-UHTCC along the interface with the concrete in all strengthened beams. The shear force capacity of the strengthened RC beams were greatly improved with the increase of the FRP grid reinforcement ratio from 0.03% to 0.2%. The maximum increase of this shear force capacity for all the strengthened beams compared to the reference beam was 59%. The construction speed can be somewhat accelerated by spraying the UHTCC layer. However, the interfacial bonding performance between the FRP-UHTCC system and the concrete is expected to be reduced in this case.

KEYWORDS

Four-point bending, RC beams, FRP grid, UHTCC, shear force capacity, strengthening.

INTRODUCTION

Due to different design requirements for resiliency and sustainability, increase of external loads due to extreme events such as earthquakes or deterioration of structural elements due to environmental effects, strengthening and rehabilitating the existing reinforce concrete (RC) structures is becoming an urgent need in the past few decades. Fibre Reinforced Polymer (FRP) as a novel material for strengthening RC structures has been widely accepted because of well-known advantages, such as light weightiness, high strength, good corrosion resistance, and higher durability (Zheng et al. 2016; Wang et al. 2014, 2015; Yao and Teng 2007). However, the bonding behaviour at the interface of FRP-to-concrete substrate may be seriously affected by some environmental factors, such as water moisture, ultraviolet (UV) light, high temperature, and fire, especially when using the epoxy resin as an interfacial adhesive (Hashemi and Al-Mahaidi 2012; Gamage et al. 2006). Therefore, some scholars attempted to replace the epoxy resin with inorganic or cementitious materials to develop new fibre composite reinforcing systems for strengthening RC structures, such as dry fibre sheets bonded with cementitious materials (Babaeidarabad et al. 2014; Xu et al. 2007), fibre-reinforced inorganic polymer (FRIP) composites (Dai et al. 2014; Ding et al. 2014) and textile reinforced mortars (TRM) (Koutas et al. 2014; Triantafillou and Papanicolaou 2006). These methods utilized the several advantages of cementitious materials. Particularly, the engineered cement-based adhesive possesses a much better material compatibility with the concrete substrate compared to the epoxy-based one.

Although using cementitious or inorganic materials to strengthen RC structures may improve their load carrying capacities and meet the functional requirements of structures under the normal service condition, some deficiencies of those materials still exist, including incompatibility of deformation between the FRP reinforcement and the
cement-based matrix, need for larger amounts of FRP reinforcement increasing cost, poor penetration ability of FRP sheet/plate, and low tensile strength and durability (Dehghani et al. 2015; Kim et al. 2014; Wu and Sun 2005). For the purpose of overcoming such drawbacks and for optimizing the strengthening method, attempts are made in the presented test program to develop a relatively new strengthening system, i.e. FRP grid in conjunction with Ultra High Toughness Cementitious Composite (UHTCC). In this FRP-UHTCC system, the UHTCC layer has two functions: 1) strengthening the RC structures, and 2) bonding the FRP grid and the concrete substrate (Zheng et al. 2015, 2016; Wang et al 2015, 2016; Chen et al. 2010). This new strengthening system is expected to provide a dual effect to the original RC structures due to the high strength of the FRP grid and the strain-hardening behaviour of the UHTCC. Meanwhile, the UHTCC as a bonding agent is expected to suppress the width of cracks and prevent the crack-induced debonding failure due to the multiple cracking behaviour of the UHTCC.

TEST PROGRAM

Design of specimens

A total of five RC beams with rectangular cross sections were tested in this study. Beam B0 was the reference without any strengthening and the other four RC beams (B1-1, B2-3, B3-5, and B4-3) were strengthened for shear with a 30 mm thick composite reinforcement layer (CRL) consisting of Basalt FRP (BFRP) grid and UHTCC. All test specimens had identical dimensions of 1800 mm in length (span length of 1700 mm), 300 mm in width and 200 mm in depth. Two ribbed steel bars (HRB335 grade) with diameter of 12 mm and four ribbed steel bars (HRB335 grade) with diameter of 20 mm were longitudinally placed at the top and bottom sides of the RC beams as compression and tension steel reinforcement, respectively, to avoid flexural failure prior to shear failure. The thickness of the concrete cover was 30 mm for both the top and bottom steel reinforcement. The transverse shear reinforcement in the form of closed steel stirrups were plain round bars (HPB235 grade) with diameter of 6 mm and spacing of 200 mm centre to centre. The common details of the cross section and steel reinforcement for the test beams are shown in Figure 4.

The investigated variables in this test program were the reinforcement ratio of the BFRP grid and the construction method of the UHTCC layer. Three thickness of BFRP grid were used to externally strengthen the UHTCC overlay: 1 mm thickness for beam B1-1, 3 mm thickness for beams B2-3 and B4-3, and 5 mm thickness for beam B3-5. In addition, two different construction methods of the UHTCC layer were applied to investigate their effect on the shear force capacity of the strengthened beams: casting for beams B1-1, B2-3, and B3-5, and spraying for beam B4-3. The material and construction properties of all test beams are summarized in Table 3.

All beams were cured for 28-days under the laboratory conditions and polished with a grinding wheel to remove the laitance and sundry on the side surfaces of the test specimens. Subsequently, sixteen steel bolts with diameter of 6 mm were symmetrically embedded into the concrete on the side surfaces of the RC beams where four of them installed at each of the two end sections and the two loading point sections. Afterwards, the BFRP grids were fixed

<table>
<thead>
<tr>
<th>Beam</th>
<th>Compressive strength [MPa]</th>
<th>UHTCC [MPa]</th>
<th>Construction method of UHTCC</th>
<th>FRP grid Types</th>
<th>Tensile strength [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>B0</td>
<td>29</td>
<td>none</td>
<td>none</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>B1-1</td>
<td>29</td>
<td>27</td>
<td>Casting</td>
<td>BFRP 0.03 (1)</td>
<td>357</td>
</tr>
<tr>
<td>B2-3</td>
<td>29</td>
<td>27</td>
<td>Casting</td>
<td>BFRP 0.12 (3)</td>
<td>386</td>
</tr>
<tr>
<td>B3-5</td>
<td>29</td>
<td>27</td>
<td>Casting</td>
<td>BFRP 0.20 (5)</td>
<td>416</td>
</tr>
<tr>
<td>B4-3</td>
<td>29</td>
<td>25</td>
<td>Spraying</td>
<td>BFRP 0.12 (3)</td>
<td>386</td>
</tr>
</tbody>
</table>

Figure 4 Common details of cross section and steel reinforcement for the test beams
on the surfaces of the test specimens through those steel bolts. As for the construction method of casting the UHTCC, a 30 mm thickness of the UHTCC layers were cast along the longitudinal directions of the beams after they were placed in wooden formworks. For the construction method of spraying the UHTCC, the CRL was formed by multiple spraying of the UHTCC mixture to the side surfaces of test specimens using an approximate thickness of 10 mm each time through a machine sprayer. It is noted that the strengthening using the FRP-UHTCC system was only applied in the bending-shear regions only, i.e. excluding the zero shear regions between the load application points of the beams, refer to Figure 4.

Test Materials

The designed strength grade of concrete for the RC beams was C30. Ordinary Portland cement (P.O 42.5 grade), fine river sand and crushed stone aggregate with maximum aggregate sizes of 5 mm and 20 mm, respectively, and water were mixed together to produce the concrete. During the casting process of the concrete, three cubic samples with dimensions of 150 mm×150 mm×150 mm were cast to determine the concrete compressive strength. The 28-day average compressive strength of concrete was 29 MPa, as shown in Table 3.

For the UHTCC, ordinary Portland cement (P.O 42.5 grade), fine silica sand, fly ash, silica fume, chopped PolyVinyl Alcohol (PVA) fibre and admixtures were mixed together to form this layer. During the casting process of the CRL, four cubic samples with dimensions of 70.7 mm×70.7 mm×70.7 mm were cast to determine the UHTCC compressive strength. The 28-day average compressive strength of UHTCC was 27 MPa for beams B1-1, B2-3 and B3-5, and 25 MPa for beam B4-3, as shown in Table 3.

Three types of steel reinforcement were used in the test program: HPB235 grade plain round bars with diameter of 6 mm for the stirrups, HRB335 grade ribbed bars with diameters of 12 mm and 20 mm for the compression (top) and tension (bottom) reinforcement of the RC beams. For these 6 mm, 12 mm, and 20 mm diameter bars, the yield strength values were 547 MPa, 580 MPa, and 605 MPa, respectively, and the ultimate strength values were 598 MPa, 641 MPa, and 683 MPa, respectively.

The BFRP grid was manufactured by Jiangsu Green Materials Vally New Material T&D Co., Ltd, China. In this grid, the transverse and horizontal fibre reinforcement was made of continuous basalt-based untwisted yarn and infiltrated into the saturated resin to form the two-way grid after the moulding resin dried. The dimensions of the BFRP grid were 625 mm×300 mm and the rigid fibre reinforcements in the BFRP grid were arranged at 50 mm centre to centre along the transverse and longitudinal directions, as shown in Figure 5. Three different thickness values (1 mm, 3 mm, and 5 mm) of the BFRP grid were used to reinforce the RC beams. The average elastic moduli of the 1 mm, 3 mm, and 5 mm thick BFRP grid were 52 GPa, 53 GPa, and 57 GPa, respectively, and the corresponding ultimate tensile strength were 357 MPa, 386 MPa, and 416 MPa, respectively.

![Figure 5 Dimensions of the used BFRP grid](image)

Test setup

All test beams were subjected to a two-point symmetric load separated by 500 mm centre to centre through a rigid distribution girder. The speed of vertical load was 0.5 kN/min before the cracking of concrete at the tensile surface of test beams, and 1.0 kN/min was set up for the subsequent loading until failure. The variation of the vertical load was collected by a loading sensor connected to a 500-kN hydraulic jack. Five LVDTs were symmetrically placed at the sections of two supports, two loading points and mid-span to measure the changes of the vertical deformations. Three strain gauges (labelled SG1, SG2 and SG3) were bonded to the surface of the stirrup at different elevations within the bending-shear regions to monitor the strain variations. The test setup and locations of the measuring points are shown in Figure 6.
TEST RESULTS

Failure modes

The typical shear-compression failure of concrete was observed for the reference beam B0 due to the eventual concrete crushing at the bending-shear region, as shown in Figure 7a. For the strengthened beams B1-1, B2-3, B3-5, and B4-3, the partial debonding of the CRL along the interface of the CRL-to-concrete was exhibited under the locations of loading points before the shear-compression failure, as shown in Figure 7b. Moreover, several fine cracks were uniformly distributed in the UHTCC layer due to the multiple cracking properties of UHTCC, as shown in Figure 7b. To further investigate the bonding performance of the CRL-to-concrete interface, one of the FRP-UHTCC layers was removed from one side surface of the failed beam, as shown in Figure 7c and 4d. The inside surface of the FRP-UHTCC was almost fully bonded with the crushed concrete, which indicated that the FRP-UHTCC played an important role in improving the shear performance of the RC beams due to the good interfacial bonding performance of the FRP-UHTCC to the concrete substrate.
**Load-deflection response**

The load-deflection responses of all test beams at the loading point and mid-span sections are shown in Figure 8. For all strengthened beams B1-1, B2-3, B3-5, and B4-3, there was a remarkable increase in the shear force capacity when the CRL was bonded to the side surfaces of the test beams, as shown in Figure 8 and Table 4. The increase of the shear force capacity for all these strengthened beams ranged between 27% and 59% compared to the reference beam B0.

![Figure 8 Load-deflection relationships of all test beams](image)

For the reference beam B0, when the applied vertical load reached 36 kN (approximately 12% of the ultimate load), a few of the flexural cracks appeared at the tension (bottom) side of the beam. With increasing of the external load, the flexural cracks gradually propagated to the top part of the test beam and continually propagated towards the locations of the loading points. When the vertical load reached 226 kN (approximately 76% of the ultimate load), the first critical diagonal crack formed followed by yielding of the stirrups within the bending-shear regions. When the vertical load approached 296 kN, the relative sliding movement of the concrete at the bending-shear regions occurred along the propagation direction of the critical diagonal crack and the shear-compression failure mode of beam B0 took place, as shown in Figure 7a.

Beam B1-1 was strengthened with 1 mm thickness of BFRP grid and UHTCC. When the load approached 65 kN (approximately 17% of the ultimate load), a few of flexural cracks were formed at the tension side of the beam in the pure bending (zero shear) region. With the load further increasing, new flexural cracks continually formed at the bending-shear regions and the previous concrete cracks gradually propagated to the top surface of the test beam. When the vertical load increased up to 206 kN (approximately 55% of the ultimate load), a few of the diagonal cracks appeared at the surface of the FRP-UHTCC layer and continually propagated to the position of the loading point. When the load approached 221 kN (approximately 59% of the ultimate load), an interfacial crack formed along the interface of the CRL-to-concrete. When the vertical load of 376 kN was applied, the partial debonding failure of the CRL occurred at the interface between the FRP-UHTCC layer and the concrete substrate. Simultaneously, the relative sliding movement of the concrete at the bending-shear regions took place, as shown in Figure 7b.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Ultimate load [kN]</th>
<th>Increase of load [%]</th>
<th>Ultimate deflection [mm]</th>
<th>Increase of deflection [%]</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>B0</td>
<td>296</td>
<td>0</td>
<td>7.92</td>
<td>0</td>
<td>SC</td>
</tr>
<tr>
<td>B1-1</td>
<td>376</td>
<td>27</td>
<td>12.70</td>
<td>60</td>
<td>PD+SC</td>
</tr>
<tr>
<td>B2-3</td>
<td>449</td>
<td>52</td>
<td>10.63</td>
<td>34</td>
<td>PD+SC</td>
</tr>
<tr>
<td>B3-5</td>
<td>471</td>
<td>59</td>
<td>11.93</td>
<td>51</td>
<td>PD+SC</td>
</tr>
<tr>
<td>B4-3</td>
<td>381</td>
<td>29</td>
<td>10.33</td>
<td>30</td>
<td>PD+SC</td>
</tr>
</tbody>
</table>

Notes: SC stands for Shear-Compression failure, PD stands for Partial-Debonding failure.

For the other three strengthened beams B2-3, B3-5, and B4-3, similar observations to those of beam B1-1 took place during the testing process. By comparing with beam B1-1, the shear force capacity of beams B2-3 and B3-5 increased by 73 kN and 95 kN, respectively, due to the increase of the BFRP grid reinforcement ratio. It should be noted that the shear force capacity of beam B4-3 was lower than that of beam B2-3 due to the construction
method of spraying the UHTCC. This suggests that indeed the construction speed maybe accelerated by using spraying UHTCC, but the interfacial bonding performance between the CRL and the concrete substrate will most likely be reduced. As a result, the interfacial bonding property deserves more research attention when using the spraying construction technique for the UHTCC as a strengthening layer.

**Stress-strain response of stirrups**

The strain distributions of stirrups using gauges SG1, SG2, and SG3 in the reference beam (B0) and the strengthened beams B1-1, B2-3, B3-5, and B4-3 within the bending-shear regions are shown in Figure 9a, 6b, and 6c, respectively. It can be observed that the strains of the stirrups in the strengthened beams were significantly decreased compared to those of the reference beam B0, due to the strengthening effect of the FRP-UHTCC. Moreover, the FRP-UHTCC strengthening system greatly improved the shear force capacity of the RC beams by sharing the tensile stresses with the internal stirrups in the bending-shear regions.

Compared to the reference beam, several finer cracks were uniformly distributed in the UHTCC layer, and the width of these cracks were greatly decreased due to the multiple cracking performance of UHTCC. It should be noted from Figure 9 that the FRP-UHTCC layer can effectively share the tensile actions of concrete and stirrups due to its good interfacial bonding performance. As a result, the toughness of UHTCC deserves more research attention when using the UHTCC as a strengthening layer.

**CONCLUSIONS**

Four strengthened beams and one reference beam were experimentally investigated to determine the shear behaviour of these RC beams when strengthened with a composite layer of FRP-UHTCC. Based on the experimental observations, the following conclusions can be drawn:
The final failure modes of the four strengthened beams were partial debonding of the CRL along the interface between the FRP-UHTCC and the concrete substrate due to the propagation of diagonal cracks of the RC beams. As expected, the typical shear-compression failure of concrete was observed in the reference beam.

The shear force capacity of the tested RC beams were greatly improved after strengthening with the composite FRP-UHTCC compared to the reference beam. In addition, the shear force capacity of these strengthened beam increased by 27% to 59% compared to the reference beam depending on the reinforcement ratio of the FRP grid, which varied from 0.03% to 0.2%.

The construction speed can be accelerated by using the spraying construction technique of the UHTCC layer. However, the interfacial bonding performance between the FRP-UHTCC and the concrete substrate is expected to be reduced with this technique compared to the usual casting construction technique.

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REFERENCES


EXPERIMENTAL STUDY ON THE MECHANICAL BEHAVIOUR OF CIRCULAR REINFORCED CONCRETE COLUMNS STRENGTHENED WITH FRP TEXTILE AND ECC

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ABSTRACT

Considering the cured surface and reinforcement layers of FRP textile and construction technology of ECC, the static axial compressive experimental program on FRP textile reinforced ECC confined circular reinforced concrete (RC) column was carried out in this paper. Based on the test results, the numerical analysis model was also established using the finite element software. The test results show that all strengthened RC column failed by the rupture of FRP textile and the ultimate compressive strength and deformation capacity of confined RC column was enhanced with the reinforced layer of FRP textile increased. After surface treatment, the bonding behavior of FRP textile and ECC interface and compatibility of those two materials were significantly improved. Besides, the composite strengthened layer combined with either smeared or sprayed ECC strengthening and BFRP textile can provide effective lateral confining stress to the strengthened RC columns and delay the buckling of longitudinal reinforcement. The analytical results show that the finite element model can simulate the mechanical process of the strengthened RC columns effectively.

KEY WORDS

FRP textile, ECC, strengthening, numerical analysis.

INTRODUCTION

Maintaining and upgrading the safety of concrete structures is a massive challenge throughout the world. The development of bonded fiber-reinforced polymer (FRP) system represents a major breakthrough in the strengthening of concrete structures over the past two decades. However, this technology suffers from a major weakness for the outdoor buildings, especially bridge engineering. For example, the degradation of bonding performance at the FRP-to-concrete interface could be occurred under the combined actions of moisture (e.g. Wang et al. 2013; Dai et al. 2014; Zhang et al. 2015), high temperature and fire (Xu et al. 2014). This is an obvious fact that the organic polymer (e.g. the epoxy resin) is usually applied as an interfacial bonding adhesive and a saturate impregnate for FRP fiber sheet.

To solve this problem, attempts have been made to replace the epoxy resin by using the cement-based material to develop new inorganic polymer or modified cement mortar for strengthening reinforced concrete (RC) members, such as dry fiber sheet bonded with cementitious materials, fiber-reinforced inorganic polymer (FRIP) composites, FRP textile reinforced mortar and concrete (TRM/TRC) and FRP grid reinforced cement mortar (e.g. Xu et al. 2011; Xun et al. 2012; Ding et al. 2014;Zheng et al. 2016). Compared with epoxy resin, a few of distinct disadvantages are still exited in the inorganic materials, such as low extensibility and easy brittleness. In this paper, a new strengthening system, FRP textile reinforced engineered cementitious composite (ECC) system is proposed to strengthen RC columns. In this technique, the high ductility of ECC is sufficiently utilized to overcome the natural shortcomings of above mentioned inorganic materials. Therefore, better material compatibility between ECC and concrete substrate can be obtained and bad actions of ultraviolet (UV), elevated temperature and moisture will not be happening. Moreover, a double confined effect is expected to be able to act on the RC column so as to enhance its load capacity and improve the deformation of that element.

EXPERIMENTAL PROGRAMS

Specimen Description

A total of fifteen RC columns subjected to the concentric compression loading were manufactured in this test
program. Among them, three non-strengthened RC columns and twelve FRP textile reinforced ECC (FRE) confined RC columns were made to study their compressive mechanical performance. All tested specimens were divided into six groups (i.e. Group RC, Group FRE1, Group FRE1E, Group FRE2E, Group FRE3E, and Group FRSE1). Each group had three identical specimens as listed in Table 1. The non-strengthened Group RC was set to the control group. The rest three Groups FRE1E, FRE2E and FRE3E were reinforced with FRE composite jacket consist of basalt fiber reinforced polymer (BFRP) textile and a layer of ECC with a 30mm fixed thickness. Group FRE1 confined with one layer of BFRP textile. One layer, two layers and three layers of epoxy-sanded BFRP textiles were used for group FRE1E, FRE2E and FRE3E, respectively. To improve the bond between BFRP textile and ECC, the epoxy adhesive resin was brushed onto the two side surfaces of BFRP fiber bundle to act as a bond layer and then 0.3~0.6 mm fine aggregate were spread onto the epoxy resin immediately to form the coarse interface. Meanwhile, the sprayed ECC was used for Groups FRES1 to investigate the influence of construction technology on the confinement of the core concrete.

All RC columns had a circular cross section with dimensions of 600 mm in length and 200 mm in diameter. Six longitudinal deformed steel rebar with 12mm diameter were uniformly distributed along the circumference as the internal steel reinforcements. The stirrups were 6mm diameter round steel rebar spaced 150 mm. Extra stirrup reinforcements were provided at two ends of RC columns to avoid the local compression failure. Details of the experimental program are presented in Table 1 and Figure 1.

Before strengthening, the surfaces of RC columns were polished firstly. A layer of ECC with approximately 10mm thick was smeared on the side surface of RC column using a smooth metal trowel. Then the epoxy-sanded BFRP textile was wrapped along the circumference of RC columns and pressed slightly into the first ECC layer, as shown in Figure 2a. After that, the cover ECC layer was smeared to protect the internal BFRP epoxy-sanded textile and form a composite reinforced jacket, as shown in Figure 2b. It should be noted the process of manufacturing the composite reinforced jacket for specimens in Groups FRES1 is similar with that of other strengthened specimens except the sprayed ECC layer.

![Figure 1 Details of exp. program](image1)

![Figure 2 FRE confined column](image2)

![Figure 3 Details of ECC specimen](image3)

<table>
<thead>
<tr>
<th>Group ID</th>
<th>Size</th>
<th>Number of specimen in one group</th>
<th>BFRP textile</th>
<th>Type of ECC</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC</td>
<td>∅200mm×600mm</td>
<td>3</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>FRE1</td>
<td>∅200mm×600mm</td>
<td>3</td>
<td>One layer</td>
<td>epoxy-sanded Smear ECC</td>
</tr>
<tr>
<td>FRE2</td>
<td>∅200mm×600mm</td>
<td>3</td>
<td>Two layers</td>
<td>epoxy-sanded Smear ECC</td>
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<tr>
<td>FRE3</td>
<td>∅200mm×600mm</td>
<td>3</td>
<td>Three layers</td>
<td>epoxy-sanded Smear ECC</td>
</tr>
<tr>
<td>FRE1E</td>
<td>∅200mm×600mm</td>
<td>3</td>
<td>One layer</td>
<td>No-sanded Spray ECC</td>
</tr>
<tr>
<td>FRE2E</td>
<td>∅200mm×600mm</td>
<td>3</td>
<td>One layer</td>
<td>No-sanded Spray ECC</td>
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<tr>
<td>FRE3E</td>
<td>∅200mm×600mm</td>
<td>3</td>
<td>One layer</td>
<td>No-sanded Spray ECC</td>
</tr>
</tbody>
</table>

**Material Properties**

The design strength grade of concrete for the RC columns was C30 according to *CHINA DESIGN SPECIFICATION OF CONCRETE STRUCTURES*. Ordinary Portland cement, fine and coarse aggregate with a maximum size of 20mm and water were mixed together to manufacture the concrete. The water cement ratio was 0.6. During the stage of casting test columns, three cube samples with dimensions of 150mm×150mm×150mm were reserved to obtain the compressive strength of concrete. The average value of 28-day compressive strength for the concrete was 37.4MPa.
Unidirectional tensile tests were conducted to investigate the mechanical properties for BFRP textile and ECC. For BFRP textile, each fiber bundle of it was 5.42mm wide, and the clear spacing between the adjacent fiber bundles was 25mm. The nominal thickness of each layer for BFRP textile was 0.67mm. The materials used in the production of ECC mixture were Ordinary Portland cement, fly ash, silica sand with a maximum particle size of 0.6mm, polycarboxylate-based high range water reducing admixture (HRWRA), and polyvinyl alcohol (PVA) fiber. To speed up the condensation of sprayed ECC, the accelerator was added into sprayed ECC. The mixed details are given in Table 2. The PVA fibers mixed in the ECC were produced by Kuraray Co., Ltd of Japan and its detailed material properties are presented in Table 3. To characterize the direct tensile behavior of ECC mixture, three dog-bone specimens (as seen in Figure 3) were used for direct tensile tests (Kuang et al. 2015; Yu et al. 2015), which were conducted under the displacement control at a loading rate of 0.25mm/min. Details of coupons for material properties were presented in Table 4.

### Table 2 Mixed Ratio of ECC

<table>
<thead>
<tr>
<th>Type</th>
<th>Cement</th>
<th>Water</th>
<th>Fly ash</th>
<th>Silica sand</th>
<th>Silica fume</th>
<th>HRWRA</th>
<th>Accelerator</th>
<th>Fiber, V_f/%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smear</td>
<td>1.0</td>
<td>0.83</td>
<td>2.53</td>
<td>0.80</td>
<td>0.10</td>
<td>0.011</td>
<td>—</td>
<td>2.0</td>
</tr>
<tr>
<td>Spray</td>
<td>1.0</td>
<td>0.49</td>
<td>0.30</td>
<td>0.26</td>
<td>0.04</td>
<td>—</td>
<td>0.02</td>
<td>2.0</td>
</tr>
</tbody>
</table>

### Table 3 Material properties of PVA fibers

<table>
<thead>
<tr>
<th>ID</th>
<th>Diameter /μm</th>
<th>Length /mm</th>
<th>Density /g/cm³</th>
<th>Elastic module /GPa</th>
<th>Tensile strength /MPa</th>
<th>Elongation /%</th>
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<tr>
<td>PVA</td>
<td>39</td>
<td>12</td>
<td>1.3</td>
<td>40</td>
<td>1530</td>
<td>7</td>
</tr>
</tbody>
</table>

### Table 4 Mechanical Properties of all materials /MPa

<table>
<thead>
<tr>
<th>Materials</th>
<th>Yield Strength</th>
<th>Ultimate Strength</th>
<th>Young’s Modulus</th>
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</thead>
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<tr>
<td>Concrete</td>
<td>—</td>
<td>37.4</td>
<td>—</td>
</tr>
<tr>
<td>BFRP textile</td>
<td>—</td>
<td>277</td>
<td>22140</td>
</tr>
<tr>
<td>BFRP textile (sanded)</td>
<td>—</td>
<td>335</td>
<td>28150</td>
</tr>
<tr>
<td>Steel bar</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Ø6</td>
<td>455.6</td>
<td>507</td>
<td>21000</td>
</tr>
<tr>
<td>Ø12</td>
<td>460.0</td>
<td>610</td>
<td>20000</td>
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<tr>
<td>ECC</td>
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<td>Smeared ECC</td>
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<tr>
<td>Sprayed ECC</td>
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<td>4.7</td>
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</tbody>
</table>

### Test Setup

For detecting the axial and longitudinal strains of FRE composite jacket, four strain gauges with a gauge length of 60mm were installed on the two opposite surfaces at the mid-height section of FRE-confined RC column. The axial deformations of all test cylinders, with or without FRE confinement, were also measured using two linear variable displacement transducers (LVDTs) installed 180° apart on the top of RC column. Besides, for the internal longitudinal steel reinforcement, 5mm-length strain gauge was attached on its surface to observe the development of axial strain. Same strain gauge was applied on the steel stirrup to obtain the hoop strain. As for BFRP textile, strain gauges were bonded on the axial and hoop fiber bundles to measure two directional strains.

Uniaxial compression tests of all RC columns were performed using a 5,000-kN capacity servo-controlled MTS machine. Each cylinder was first loaded to around one-third of its unconfined capacity to check the alignment. The axial load was then increased at a load rate of approximately 2.4kN/s until the specimens failed. During the test, all strain gauges and LVDTs data were recorded using a data acquisition system capable of reading one set of data per second.

### RESULTS AND DISCUSSION

#### Failure Mode

Typical failures of the test specimens are shown in Figure 4. The control specimen failed with concrete crushing after the longitudinal reinforcing steel rebar yielded. A plurality of vertical cracks was distributed on surface of the column body. Relatively, all reinforced specimens failed with the circumferential tensile rupture of cured fiber bundles at the vertical main fracture of FRE jacket. Meanwhile, failure was usually accompanied by many micro cracks rather than deboning over a large portion of the cylinder height. This indicated that the characters of ECC, e.g. multiple cracking, high ductility and good bond behavior with substrate concrete, were obtained during the test progress.
It was a remarkable fact that the composite jacket with sanded textile ruptured as an integral in group FRE1E–FRE3E (Figures 4c–4e). Comparatively, the test result showed that an obvious interface slipping phenomenon occurred between ECC and textile in group FRE1 (Figure 4b).

Figure 4 Failure Modes of Specimens

Load-Displacement Responses

The load-displacement responses of all specimens are shown in Figure 5. In the rising stage, the curve slope of confined columns was similar to those of control specimens (Group RC), which indicated that FRE composite jacket had limited resistant effect to core concrete when load was small. The growth of loading rate caused vertical cracks in ECC and then resulted in the stiffness of column reduced gradually. After longitudinal steel rebar yielded, axial stiffness decreased more significantly. The increase of hoop deformation caused rapidly expanding cracks. Therefore, the confined columns damaged with BFRP textile rupture finally.

Figure 5 Load-Displacement Curves

Compare to the control specimens, the average cracking load (near 950kN) of Groups FRE1E–FRE3E improved by 73%. Besides, the average ultimate load of Group FRE1E, FRE2E and FRE3E increased by 44.6%, 49% and 56.4%, respectively. This indicated that the ultimate compressive strength of confined columns could be enhanced, to an extent, by improving the textile layers. It was worth mentioning that the average ultimate compressive strength of Group FRSE1 increased by 62.6%, which indicated that FRE composite jacket constructed by sprayed ECC, could provide effective confinement to core concrete. Table 5 shows the details of cracking and ultimate loads for all test specimens.

Stress-Strain Responses

The axial compressive stress-strain curves of FRE1E, FRE2E, FRE3E and RC were presented in Figure 6. Here, axial reinforcement strain was used to present the axial deformation of specimens in this paper. As the figure shows, the curve can be divided by two parts. Similar to those of conventional FRP-confined concrete with a sufficient level of FRP confinement (Teng et al. 2007), the stress-strain curves of FRE-confined concrete from the present tests also exhibited a monotonically ascending bilinear shape with rapid softening in a transition zone around the stress level of unconfined concrete strength. Both the compressive strength and the ultimate axial strain were significantly enhanced. However, be worth mentioning, the curve slope of part two was gentler than that of conventional FRP-confined.
Table 5 Summary of Test and Analytical Results

<table>
<thead>
<tr>
<th>Specimen notation</th>
<th>Experimental Result/kN</th>
<th>Simulated Result/kN</th>
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<tr>
<td></td>
<td>Crack load P&lt;sub&gt;cr&lt;/sub&gt;, Average P&lt;sub&gt;cr&lt;/sub&gt;, Ult. load P&lt;sub&gt;u&lt;/sub&gt;,</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>P&lt;sub&gt;cr&lt;/sub&gt;, P&lt;sub&gt;u&lt;/sub&gt;, P&lt;sub&gt;cr&lt;/sub&gt;, P&lt;sub&gt;u&lt;/sub&gt;,</td>
<td>P&lt;sub&gt;cr&lt;/sub&gt;, P&lt;sub&gt;u&lt;/sub&gt;,</td>
<td></td>
</tr>
<tr>
<td></td>
<td>P&lt;sub&gt;cr&lt;/sub&gt;, P&lt;sub&gt;u&lt;/sub&gt;, P&lt;sub&gt;cr&lt;/sub&gt;, P&lt;sub&gt;u&lt;/sub&gt;,</td>
<td>P&lt;sub&gt;cr&lt;/sub&gt;, P&lt;sub&gt;u&lt;/sub&gt;,</td>
<td></td>
</tr>
<tr>
<td>RC-1</td>
<td>545.0</td>
<td>547.9</td>
<td>780.0</td>
</tr>
<tr>
<td>RC-2</td>
<td>526.1</td>
<td>735.3</td>
<td>785.8</td>
</tr>
<tr>
<td>RC-3</td>
<td>572.5</td>
<td>842.1</td>
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<td>FRE1-1</td>
<td>743.6</td>
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<td>FRE2E-1</td>
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<tr>
<td>SD</td>
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</tbody>
</table>

The typical hoop strain responses of ECC and BFRP textile are shown in Figure 7. As presented, both of them were very little when the axial compressive stress lowered than 15MPa. The strain increased rapidly as the axial compressive stress reached to 35MPa. Obviously, the circumferential strain of sanded textile and ECC were basically the same throughout the test progress. Therefore, there was no interface slip occurred between them. On the contrary, the hoop strain of ECC was obvious much larger than that of textile in FRE1-1. This was consistent with the failure mode.

FINITE ELEMENT MODELING

Finite element modeling (FEM) of wrapped as well as control specimens were carried out in order to analyze the experimental results. ABAQUS, a general-purpose nonlinear finite element program, was employed for the numerical simulation. The concrete damaged plasticity model was used to simulate concrete (CODE FOR DESIGN OF CONCRETE STRUCTURES). Based on the axial tension test, plastic model was used to assign material properties to ECC and steel. As shown in Figure 8, σ<sub>cr</sub>, ε<sub>cr</sub>, σ<sub>cu</sub>, ε<sub>cu</sub> are cracking stress/strain and limiting stress/strain of ECC. Besides, BFRP textile simulated with linear elastic material, and the compressive behavior of longitudinal textile was not considered in this simulation. Static general analysis method was used to calculate the model and there was no interface slipping occurred between ECC and textile. Details of numerical model are shown in Figure 9.

Figure 8 ECC tension response
Figure 9 Numerical Model
Figure 10 Comparison of Failure Mode
Figure 11 Comparison of experimental and simulated results

As shown in Figure 10, the PEEQ (equivalent plastic strain) result of simulated specimen was similar to that of experimental cylinder for selected specimen FRE1E. The destruction area of FRE composite jacket was focus on the center of column, and the angle between cracking direction and specimen axis was about 45°. The result showed that the simulated specimens also damaged with BFRP textile rupturing, which was matched with the experimental result.

The compared results of test and simulation were shown in Figure 11 and Table 5. As seen in Table 5, the average ratios of cracking load and ultimate load between simulation and experimental result were 0.955 and 1.032 respectively. And the standard deviations were only 0.068 and 0.048 respectively. This verified the feasibility of the finite element modeling.

CONCLUSION

After surface treatment of textile, the bonding behavior of BFRP textile-ECC interface was significantly improved. Either smeared or sprayed ECC reinforced with BFRP textile could provide effective lateral confining stress to the core RC columns and delay the buckling of longitudinal reinforcement.

The effectiveness of the proposed new confined method was validated through a series of experimental programs. The test results indicated the loading capacity and axial deformation of the strengthened RC column were improved obviously.

The analytical results show that the finite element model can simulate the mechanical process of strengthened RC columns effectively.

REFERENCES


THE BEHAVIOR BETWEEN CFRP GRID AND STIRRUP IN RC BEAMS STRENGTHENED BY CFRP GRID AND SPRAYED MORTAR

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2 Department of Civil Engineering, University of Transport and Communications, Viet Nam

ABSTRACT
This work aims at studying on reinforced concrete (RC) beams shear-strengthened by Carbon Fibre-Reinforced Polymer (CFRP) grid and sprayed mortar. Clearly understanding the behaviour between stirrup and CFRP grid is necessary for evaluating the effective of the strengthening work. Two RC beams were fabricated; the control beam was not strengthened while the other was shear-strengthened by CFRP grid and sprayed mortar. The four-points bending tests were conducted until both of beams were failure. The main data of the experimental program was presented and analysed to show the interaction between stirrup and CFRP grid. The experimental results indicated that the shear strength of RC beam was significantly increased by the CFRP grid and sprayed mortar. Bonding between concrete and sprayed mortar is one of the most important factors that influence the effective reinforcement. Cracks occurring in RC beam after the plastic period considerably affect the behaviour between stirrup and CFRP grid.

KEYWORDS
RC beams, shear-strengthening, CFRP grid, sprayed mortar, stirrup.

INTRODUCTION
In recent decades, the reparation of existing structures such as building, bridges, etc., have been amongst the most importance challenges of civil engineering. The primary reason for strengthening of structure includes upgrading of its resistance to withstand underestimated loads, restoration of lost load carrying capacity due to corrosion or other types of degradation. Carbon fibre reinforced polymer (CFRP) composites are widely used for strengthening concrete structures because of their advantages over conventional strengthening methods. And strengthening with externally bonded CFRP fabric has shown to be applicable to many kinds of structures. In recently years, many concrete structures have been strengthened by CFRP grid and in many specific cases it shows the advantage when comparing to CFRP sheets or plates.

Many previous experimental researches have been carried out over the past decade to study on the performance of concrete beams strengthened in shear with externally bonded FRP composites. I.A. Bukhari [1] has reviewed existing design guidelines for strengthening continuous beams in shear with CFRP sheets and proposes a modification to Concrete Society Technical Report. On the other hand, Wen Yong Chen [2] studied the shear behaviour of RC beam with FRP grid and supposed that RC beams strengthened with FRP grid have good shear behaviour both increasing shear capacity and controlling of the crack width. In research of Rui Guo [3] examined the effect of shearing capacity of RC beams with haunch by PCM shotcrete method with CFRP grid and adhesive properties of the reinforced interface between PCM and existing concrete was investigated and analysed. All these articles have not focused on the behavior between stirrup and CFRP grid. The main purpose of this research was to examine the effective of CFRP grid in improving the shear strength by analyzing strain of stirrup in non-strengthened RC beam and the strain of CFRP grid in the shear strengthened RC beam. This research also illustrates the behaviour between stirrup and CFRP grid in simply-supported singly-reinforced concrete beams. In fact, shear resistance capacity of RC beam gradually reduced by the time of exploiting and corrosion is one of the most important reasons causing loss of concrete-strength and reduction of sectional area of reinforced bars. In order to solve this problem, this experimental program studies on two RC beams that have same dimensions, concrete, main reinforcement bars but different in diameter of stirrup bars. The shortages of shear resistance capacity would be added by CFRP grid and sprayed mortar.
EXPERIMENTAL PROGRAM

Material

The test experiment consisted of casting and testing two beams. Two RC beams were fabricated using the ready mix concrete with the compressive strength of 34.1 N/mm$^2$ (Table 2) and the aggregate G1, G2 (with the range of finenesses of 1±1@ and 3±1%, respectively). In the mix proportion of concrete, high early strength Portland cement was used and water-cement was 0.556 with the addition of admixtures (Table 1).

Sprayed mortar was polymer-modified cement with the proportion of premixed mortar, shot polymer and water was 25kg, 1.21kg and 4.1kg, respectively. In order to increase the adhesion of the interface, an epoxy primer was applied to the surface of concrete before spraying mortar.

D32 was the tension reinforcing bar and D6, D10 were the stirrups and reinforcing bar in the compressive zone. CFRP-CR8 with grid spacing of 100x100mm was used to enhance shear strength. The mechanical properties of these materials are given in the Table 3.

Test specimens

The control beam had not been any strengthened while the remain had been shear strengthened with CFRP grid and sprayed mortar. Both of beams have the original size was 200x500x2750mm. The control beam, name RC beam 1 (RC1), is normal RC beam used D32 bar as the tension reinforcements, the compression reinforcements and the stirrups was D10, the stirrup spacing was 200mm. The remain, RC beam 2 (RC2), was strengthened by CFRP grid and sprayed mortar. CFRP grid was placed along the beam web and the sprayed mortar to create a 20mm additional layer on both sides of RC beam 2. With the assumption that the shear strength in this beam was lost due to corrosion, the section area of the stirrups reduced. In RC beam 2, D6 bar was used as stirrup which is smaller type in the RC1 (D10). Electrical resistance strain gauges were embedded to reinforced bars, stirrups, concrete and CFRP to monitor the variation of stress and strain in two beams (Figure 6 and Figure 7).
Casting RC beams

Firstly, the substrate concrete beams were cast in wooden molds and the curing by moisture-retaining material. Eight days after making the substrate beams RC2, their web’s surface was sand blasted. Four days later, CFRP grid was place on the both sides of beam using steel bold anchors.

In the next day, an epoxy primer was applied and the repair mortar was sprayed after epoxy layer had dried. Finally, a curing compound was sprayed on the mortar surface. The tests were carried out at the age of 27th and 29th day of the substrate beams. Figure 3 and Figure 4 shows the specimen’s fabrication procedure.

Test procedure

The four-points bending test was conducted with 2 beams (Figure 5). During test, when appearing the first flexural crack and the first diagonal crack, the specimens were unloaded to mark the cracks and take photographs. After that continually increase the load until 2 beams were failed. During the test, the failure processes were monitored by strain gauges embedded to concrete, reinforce bars and CFRP grid.

RESULTS AND DISCUSSIONS

Effectiveness of CFRP grid and sprayed mortar

The loading results of each test specimen are listed in Table 4. Design values were calculated according to the Standard Specifications for Concrete Structures-Design [6]. The flexural crack appeared when the flexural cracking strength of material exceeds. After that, if the stress reached the shear strength of concrete, shear cracks would occur. The design shear capacity of RC beam 1 is the sum of concrete and reinforcement while it of RC beam 2 is the sum of all components as concrete, reinforcement and CFRP grid.

<table>
<thead>
<tr>
<th>No</th>
<th>Beam</th>
<th>Maximum load (kN)</th>
<th>Design</th>
<th>Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>RC beam 1</td>
<td>354</td>
<td></td>
<td>690</td>
</tr>
<tr>
<td>2</td>
<td>RC beam 2</td>
<td>697</td>
<td></td>
<td>757</td>
</tr>
</tbody>
</table>
According to Table 4, the maximum load of RC2 (757kN) is higher 9.7% than it of RC1 (690kN). Noted that, cross sectional area of stirrup bar had been reduced from D10 to D6, equivalent to 66% of sectional area reducing. It shows that, CFRP grid had positive effect in shear strengthening. The composition of concrete, rebar, mortar and CFRP grid worked well together.

An unreasonable value of maximum load design of the two beams. Test load of RC1 was much more than design load due to the safety ratio in Standard Specifications [6]. Applying the formula for calculating shear strength of steel bar to CFRP grid was not appropriately, some coefficients in the formula was not reasonable with FRP.

Figure 6 and Figure 7 show locations of strain gauges embedded on rebar, concrete, and CFRP grid in RC beam 1 and RC beam 2. The cracks occurred during the test also were described in the Figure 6 and Figure 7, the ultimate cracks (the red curves) were largest crack at ultimate failure state of each beam.

In order to understand more clearly about the effectiveness of CRFP grid, comparing the different in strain between stirrup in RC1 and RC2 is shown in Figure 8.

![Figure 15 Location of strain gauges and experimental cracks in RC beam 1](image1)

![Figure 16 Location strain gauges and experimental cracks in RC beam 2](image2)

![Figure 17 Load versus strain curves of stirrups in RC beam 1 and RC beam 2](image3)
Table 9 Comparison of strain values on stirrup in RC beam 1 and RC beam 2

<table>
<thead>
<tr>
<th>No</th>
<th>Load (kN)</th>
<th>RC BEAM 1 (μ)</th>
<th>RC BEAM 2 (μ)</th>
<th>RC2/RC1</th>
</tr>
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<tr>
<td></td>
<td></td>
<td>S1</td>
<td>S2</td>
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<td>2</td>
<td>100</td>
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</tr>
<tr>
<td>15</td>
<td>750</td>
<td></td>
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</tbody>
</table>

The initial purpose of strengthening by CFRP grid is to take over the losing shear strength caused by reducing cross sectional area of stirrup bars. Based on Figure 8 and Table 5 the effectiveness of strengthening was better than expected.

In general, strains on stirrup S1, S2, S3, S4 (location of each stirrup showed in Figure 6, Figure 7) in RC1 were significantly longer than in RC2. The ultimate strain on S2, S3 in RC2 is over 0.015 while in RC1 is over 0.025 (Figure 8).

When the load was under 400kN, the ratio of strain in RC2 to strain in RC1 at a same stirrup is around 0.5 (Table 5). At the period of load from 400kN to 550kN, values of stirrups S2, S3, S4 in RC2 are even higher than in RC1. This is because 2 reasons: The first, strain values of S2, S3 in RC2 were from 1248x10^{-6} and 1822x10^{-6}. With values of Young’s Modulus of Steel is 2x10^5 N/mm2 (Table 3) and Yield strength of Stirrup is 417 N/mm^2 (Table 2), stirrup would become plastic at the value 2,085x10^{-6}. In this period, stress in Stirrup no longer increase. The second, in the RC beam 2, stirrup steel was D6 bars, smaller than 66% in cross sectional area than in RC beam 1. In addition, many cracks developed around the position of S2, S3, and S4 (Figure 7) caused stress concentration.

When the load is over 400kN, S4 in RC2 higher than in RC1 (Figure 7), the ultimate crack cross over the position of stirrup S4 (RC2). There was stress concentration phenomenon here.

**Behaviour between CFRP grid and stirrup in RC beam 2**

![Stirrup S4 and CFRP grid G35](image1)

![Stirrup S3 and CFRP grid G30](image2)

Figure 18 Load versus strain curves on stirrups and CFRP grid in RC beam 2

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Bonding between concrete and sprayed mortar is one of the most important factors influence the effectiveness of reinforcement. The behaviour between stirrup and CFRP grid at the same positions in RC beam 2 reflects the bonding between concrete and sprayed mortar. Strain gauges on CFRP grid G4, G14, G30, G35 and Strain gauges on stirrup bars: S1, S2, S3, S4 were at respectively same positions in the RC beam 2. Locations of these strain gauges are shown in Figure 16. Those data of the bending test are listed in Table 10. Figure 18 illustrates the load versus strain curves of stirrups and CFRP grid in RC beam 2.

Regarding to the strain curves when the load is from 300kN to 550kN, according to the collected data in Table 10 and graph in Figure 18 it is supposed that: When the load is over 350kN, Strain values of stirrup were higher than CFRP (except stirrup S1), comparison ratio of S3/G30 and S4/G35 is higher than S2/G14. At the load of more than 700kN, different in strain between stirrup S4 and CFRP grid G35 became larger. Especially, at the ultimate stage, strain of S4 was 5 times higher than G35 (4.9 times at load of 750kN). Strain gauges at the same position differed strongly. This tendency proved that, these materials (CFRP grid and stirrup) had no longer worked together and bonding between concrete and mortar was able to be damaged. These because of following reasons: Firstly, when the strains of stirrups S2, S3, S4 was over 2,085x10^-6 (this value has been calculated in previous section), stirrups became plastic state, deformation of stirrups increase rapidly after this point. Secondly, when increasing the load from 320kN to 425kN, the cracks occurred and developed deeper and wider, compression area of cross section reduced, and most of them located on the same position with S2, S3, S4 (Figure 16). Once the crack crossed the locations of stirrups and CFRP grid, adhesion of mortar to concrete surface was also damaged. About some unexpected values: when the load was low (under 200kN), strain value of G30 (too small or under 0) on CFRP grid would not be accurate. Stirrup S1, at the load over 400, strain value of S1 smaller than CFRP because, stirrup S1 worked in elastic state in whole stage of test, and there was almost no crack occurred on this location.

CONCLUSIONS

The experimental program was conducted to study on shear strengthening RC beams by CFRP grid and sprayed mortar. Base on the data and the analysis, effect of strengthening and interaction between CFRP grid and stirrup has been more clearly understand. Two following conclusions have been drawn:

The first, the test show that using CFRP grid and sprayed mortar can enhance a significant effective in shear strengthening to RC beams. The shortage of shear resistance capacity in RC beam 2 due to stirrup bars had a reduction of cross sectional area can be added by CFRP grid. With the assumption that cross sectional area of stirrup substantial reduced (66% in the test), RC beam was reinforced by CFRP grid and sprayed mortar, and shear bearing capacity of RC beam increased more than expected (9.7% much more than the control beam).

The second, the behaviour between CFRP grid and stirrups reflects the bonding between concrete and sprayed mortar. It is one of the most important factors that influence the effective reinforcement. According to results from the experiment, when increasing the load, variations of stirrup strain and CFRP grid strain at a same position in strengthened RC beam normally have similar tendencies. Cracks occurring after the plastic period of RC beam considerably affect the interaction between stirrup and CFRP grid, strain values of stirrup and CFRP grid had

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unexpectedly changed in this period. In general, it is shown that the composition of concrete, steel rebar, mortar and CFRP grid have worked properly.

From these above comments about the experiment work, it is supposed that shear-strengthening RC beams by CFRP grid and sprayed mortar provides valuable reinforcement.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the financial support provided by the Asian Human Resources Fund from Tokyo Metropolitan Tokyo.

REFERENCES

TEXTILE REINFORCED MORTAR VERSUS FRP FOR CONFINED CONCRETE: BEHAVIOUR AT ELEVATED TEMPERATURES

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ABSTRACT

Despite the clear advantages of confinement of concrete structural elements with fibre-reinforced polymers (FRPs) for strength and deformability enhancement, concerns as to their performance at elevated temperature, or in fire, remain. The results of a series of elevated temperature experiments on FRP and textile reinforced mortar (TRM) strengthening systems for confinement of circular concrete columns are presented. The behaviour and effectiveness of the respective confining systems is studied up to temperatures of 400°C. A total of 24 concrete cylinders were wrapped in the hoop direction with different amounts of FRP or TRM, heated to steady-state temperatures between 20 and 400°C, and loaded to failure in concentric axial compression under a steady-state thermal regime. The results indicate that the effectiveness of the FRP confining system bonded with epoxy decreased considerably, but did not vanish, with increasing temperatures, in particular within the region of the glass transition temperature of the epoxy resin/adhesive. Conversely, the TRM confining system, bonded with inorganic mortar rather than epoxy, demonstrated superior performance at 400°C as compared against tests performed at ambient temperature. Additional research is needed to better understand the reasons for this.

KEYWORDS

FRP, textile reinforced mortar, TRM, strengthening, high temperature, fire, confinement.

INTRODUCTION

A popular application of FRP is strengthening concrete columns by confinement with FRP in the hoop direction (Bisby et al. 2011). However, the performance of FRP systems at elevated temperatures is potentially problematic since much of their strength, stiffness, and bond properties are lost at temperatures that are rapidly exceeded in building fires (Chowdhury et al. 2011). Carbon fibres used in FRP systems are capable of resisting temperatures of more than 800°C; however, epoxies used to bond the fibres and adhere FRP systems to concrete lose a considerable proportion of their mechanical properties at temperatures as low as 60°C-82°C (ACI 2008). These reductions in mechanical properties can be expected in the region of the epoxy’s glass transition temperature ($T_g$) as it changes from hard and brittle to soft and plastic. Testing has been carried out previously to investigate the performance of FRP confining systems for concrete columns during standard fire exposures (e.g. Chowdhury et al. 2007) and during both transient and steady-state heating to elevated temperatures (Rickard et al. 2013). This testing has shown that considerable (approximately 50%) loss of effectiveness of the FRP strengthening system occurs at temperatures as low as 15°C below the $T_g$ of the epoxy adhesive, which is thought to be due to reductions in the tensile strength of the FRP wraps at these temperatures. To alleviate the reduced effectiveness of FRP systems at high temperatures, a novel textile-reinforced mortar (TRM) has been proposed for strengthening of reinforced concrete (RC) columns (Bournas et al. 2007). In an effort to develop strengthening systems with enhanced performance at elevated temperatures, this paper presents initial research aimed at investigating the performance and effectiveness of both FRP and TRM confining materials of concrete at elevated temperatures, such as would be experienced during a fire, when the confining system is active under sustained load; this was done by testing concrete cylinders with different amounts of FRP or TRM confinement at various temperatures. The main objective was to develop an initial understanding of the confining mechanisms at elevated temperature and to suggest defensible limiting temperatures for FRP and TRM strengthening systems in fire.
EXPERIMENTAL PROGRAM

Thirty tests were performed on normal strength concrete cylinders that were loaded at both ambient and elevated temperatures. All tests were performed under a steady state thermal regime. Since this is the first ever study of its kind; a goal of the study was to identify aspects for further investigation. Details of the experimental program are given in Table 1. All tests were on 100mm diameter, 200mm tall concrete cylinders; this was chosen as it allowed test specimens to fit within a bespoke environmental chamber fitted within a 600kN materials testing frame; it is noteworthy that a size effect may be relevant to the performance of both FRP and TRM wrapped cylinders, and additional research is needed on this topic. Parameters varied within the testing program included:

Type and amount of confinement: Six cylinders were tested without confinement to determine the unconfined concrete properties and to determine the reductions in concrete properties caused by heating to the maximum exposure temperature used (400°C). Twelve cylinders were wrapped with FRP (six with a single layer and six with three continuous layers). The fibres used were carbon textile reinforcement with an open weave and a weight of 220g/m². The fibres’ manufacturer datasheets (Bentonex RC225-TH12) gave a tensile strength and modulus of elasticity of the carbon fibres was 4800MPa and 225GPa, respectively, with a nominal thickness of 0.062mm (Bentonex 2016) (trade names given only for factual accuracy). The fibres were saturated and bonded using Sikadur 330 epoxy adhesive system (Sika 2016). The TRM system also used Bentonex RC225-TH12 carbon fibre textile reinforcement. This was bonded with a cementitious mortar based on an inorganic dry binder consisting of cement and polymers at a ratio of 8:1 by weight, ISOMAT (2016). The water-binder ratio in the mortar was 0.23:1 by weight, resulting in plastic consistency and good workability. The hoop overlap length was chosen as 100 mm.

Thermal Exposure: Temperatures were chosen in the range of the glass transition temperature \(T_g\), as determined from dynamic mechanical analysis and the decomposition temperature \(T_d\), as determined from thermogravimetric analysis) of the epoxy adhesive/saturant. The epoxy resin used in the current study has a \(T_g\) of 58°C based on tan delta peak from DMA testing. Samples were heated without any applied load at a heating rate of 10°C/min until the target temperature was reached, and all samples were then held at the target testing temperature for 60 minutes before testing (to assure a uniform sample temperature).

Table 11 Details of the Experimental Program

<table>
<thead>
<tr>
<th>No.</th>
<th>Wrapping</th>
<th>Matrx/Matrix</th>
<th>Exposure Temp. (°C)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
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<td>--</td>
<td>--</td>
<td>20</td>
<td>Control tests on unconfined concrete at ambient (with repeat test)</td>
</tr>
<tr>
<td>2</td>
<td>--</td>
<td>--</td>
<td>100</td>
<td>Control tests on unconfined concrete at elevated temperature</td>
</tr>
<tr>
<td>3</td>
<td>Single layer of FRP</td>
<td>Epoxy</td>
<td>20</td>
<td>FRP confined concrete at ambient</td>
</tr>
<tr>
<td>4</td>
<td>Single layer of FRP</td>
<td>Epoxy</td>
<td>100</td>
<td>Temperatures in the range of (T_g)*</td>
</tr>
<tr>
<td>5</td>
<td>Single layer of FRP</td>
<td>Epoxy</td>
<td>150</td>
<td>Temperatures well above (T_g)*</td>
</tr>
<tr>
<td>6</td>
<td>Single layer of FRP</td>
<td>Epoxy</td>
<td>200</td>
<td>Temperature in the range of (T_g)*</td>
</tr>
<tr>
<td>7</td>
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<td>Epoxy</td>
<td>400</td>
<td>Temperature in the range of (T_g)*</td>
</tr>
<tr>
<td>8</td>
<td>Single layer of TRM</td>
<td>Mortar</td>
<td>20</td>
<td>TRM confined concrete at ambient</td>
</tr>
<tr>
<td>9</td>
<td>Single layer of TRM</td>
<td>Mortar</td>
<td>100</td>
<td>Single layer of TRM at various temperatures</td>
</tr>
<tr>
<td>10</td>
<td>Single layer of TRM</td>
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<td>150</td>
<td>Repeat test at 400°C</td>
</tr>
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<td>11</td>
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<td>Repeat test at 400°C</td>
</tr>
<tr>
<td>12</td>
<td>Three layers of FRP</td>
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<td>20</td>
<td>FRP confined concrete at ambient</td>
</tr>
<tr>
<td>13</td>
<td>Three layers of FRP</td>
<td>Epoxy</td>
<td>100</td>
<td>Repeat test at ambient</td>
</tr>
<tr>
<td>14</td>
<td>Three layers of FRP</td>
<td>Epoxy</td>
<td>150</td>
<td>Temperature in the range of (T_g)*</td>
</tr>
<tr>
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<td>Temperatures well above (T_g)*</td>
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<tr>
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<td>20</td>
<td>TRM confined concrete at ambient</td>
</tr>
<tr>
<td>18</td>
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<td>100</td>
<td>Three layers of TRM at various temperatures</td>
</tr>
<tr>
<td>19</td>
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<td>Mortar</td>
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<td>Repeat test at 400°C</td>
</tr>
<tr>
<td>20</td>
<td>Three layers of TRM</td>
<td>Mortar</td>
<td>200</td>
<td>Repeat test at 400°C</td>
</tr>
</tbody>
</table>

* \(T_g\) – epoxy glass transition temperature; ** \(T_d\) – epoxy decomposition temperature
All samples were tested in an Instron 600LX testing frame with an integrated environmental chamber at a loading rate of 1mm/min (crosshead displacement) until failure. Axial and hoop strains were measured using image correlation analysis via digital images captured every 5 seconds during testing with post-testing analysis performed in GeoPIV software (White et al. 2003). Full details of the image analysis technique are avoided here but have been presented for similar testing by Rickard et al. (2013). Temperatures were monitored during testing by a single thermocouple for air temperature (placed next to the sample) and a single thermocouple mounted on the surface of the sample at mid-height. Testing occurred six months after casting the concrete.

**EXPERIMENTAL RESULTS**

Summary plots showing the results of all tests listed in Table 1 are given in Figure 1, and numerical summaries of the tests are given in Table 2.

<table>
<thead>
<tr>
<th>No.</th>
<th>Wrapping</th>
<th>Exposure Temp. (°C)</th>
<th>Temp. at Failure Temp. (°C)</th>
<th>Peak Load (kN)</th>
<th>Peak Stress (MPa)</th>
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<td></td>
<td>100</td>
<td>99 96</td>
<td>107.2</td>
<td>13.6</td>
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<tr>
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<td>150 139</td>
<td>111.4</td>
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<td>--</td>
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<td></td>
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<td>200 189</td>
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<td>400 362</td>
<td>127.3</td>
<td>16.2</td>
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<tr>
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<td>32.5</td>
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</tr>
<tr>
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<td></td>
<td>80</td>
<td>76 74</td>
<td>220.2</td>
<td>28.0</td>
<td>Rupture</td>
</tr>
<tr>
<td>8</td>
<td></td>
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<td>94 93</td>
<td>161.1</td>
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</tr>
<tr>
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<td>148 145</td>
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<td>10</td>
<td></td>
<td>200</td>
<td>196 186</td>
<td>179.6</td>
<td>22.9</td>
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</tr>
<tr>
<td>11</td>
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<td>394 370</td>
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<td>21.1</td>
<td>Adhesive</td>
</tr>
<tr>
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<td></td>
<td>20</td>
<td>20 20</td>
<td>243.1</td>
<td>31.0</td>
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</tr>
<tr>
<td>13</td>
<td></td>
<td>100</td>
<td>94 93</td>
<td>182.3</td>
<td>23.2</td>
<td>Rupture</td>
</tr>
<tr>
<td>14</td>
<td></td>
<td>150</td>
<td>146 143</td>
<td>192.8</td>
<td>24.5</td>
<td>Mixed</td>
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<tr>
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<td>200</td>
<td>195 186</td>
<td>201.4</td>
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</tr>
<tr>
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<td>222.5</td>
<td>29.0</td>
<td>Rupture</td>
</tr>
<tr>
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<td></td>
<td>100</td>
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<td>182.3</td>
<td>23.2</td>
<td>Rupture</td>
</tr>
<tr>
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<td>20</td>
<td>20 20</td>
<td>491.6</td>
<td>62.6</td>
<td>Rupture</td>
</tr>
<tr>
<td>20</td>
<td></td>
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<td>93 91</td>
<td>367.4</td>
<td>46.8</td>
<td>Rupture</td>
</tr>
<tr>
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<td>147 139</td>
<td>328.9</td>
<td>41.9</td>
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</tr>
<tr>
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<td></td>
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<td>195 188</td>
<td>328.1</td>
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<td>Adhesive</td>
</tr>
<tr>
<td>23</td>
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<td>384 371</td>
<td>302.2</td>
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<td>Adhesive</td>
</tr>
<tr>
<td>24</td>
<td></td>
<td>20</td>
<td>20 20</td>
<td>494.0</td>
<td>62.9</td>
<td>Rupture</td>
</tr>
<tr>
<td>25</td>
<td>Three layers of TRM</td>
<td>150</td>
<td>146 141</td>
<td>300.1</td>
<td>36.9</td>
<td>Rupture</td>
</tr>
<tr>
<td>26</td>
<td></td>
<td>200</td>
<td>194 186</td>
<td>317.8</td>
<td>40.5</td>
<td>Rupture</td>
</tr>
<tr>
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<td></td>
<td>400</td>
<td>396 382</td>
<td>353.1</td>
<td>45.0</td>
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</tr>
<tr>
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<td></td>
<td>383</td>
<td>362 351</td>
<td>353.1</td>
<td>45.0</td>
<td>Mixed</td>
</tr>
</tbody>
</table>

* "Rupture" refers to FRP failure in hoop tension, “adhesive” refers to loss of confinement by debonding, “Mixed” refers to failure which displayed a combination of rupture/adhesive failure.

**Unconfined Concrete Cylinders**

Six unwrapped (plain) concrete cylinders were tested. Two were tested to define ambient strength, and the remaining four were tested at selected elevated temperatures (100 °C, 150 °C, 200 °C and 400 °C) in order to assess the effects of elevated temperature on the strength and stiffness of the unconfined concrete; such that any strength reductions observed for the FRP and TRM confined specimens could be attributed (or not) to reductions in the effectiveness of the confining mechanism. The average ambient unwrapped concrete strength was 16.1 MPa. The concrete strength was reduced slightly, by 15% and 12% at 100 °C and 150 °C, respectively, before recovering to a strength 5% above that at ambient at 200 °C. At 400 °C there was no obvious reduction in compressive strength. Whilst its difficult to draw definitive conclusions regarding the effects of elevated temperature exposure on the plain concrete cylinders due to the small number of specimens tested, it is noteworthy both that these test data contradict the widely accepted concrete strength reductions with temperature suggested in the Structural Eurocodes.
(CEN, 2004) – which indicates an expected compressive strength reduction of between 15 and 20% at 400°C, depending on the aggregate mineralogy – and that similar strength trends seem to underlie the responses of the FRP and TRM confined specimens (discussed below). The lack of reduction in compressive strength of the concrete at 400°C suggests that additional research is warranted in this area.

The lack of reduction in compressive strength of the concrete at 400°C suggests that additional research is warranted in this area.

Tests on FRP Confined Concrete

Figure 1 and Table 2 show a clear trend of reducing FRP confined concrete strength with increased steady state exposure temperature; although with such a small number of samples it is difficult to clearly distinguish if this is attributed to concrete thermal damage effects and/or loss of FRP confinement effects. Whilst the results for FRP confined cylinders are similar to those previously reported by Rickard et al. (2013), additional testing with a larger number of cylinders will be required to have greater statistical confidence in the results obtained.

It is clear that exposure temperatures above \( T_g \) cause considerable reductions in the strength of FRP confined concrete cylinders. It is also clear, however, as previously reported by Rickard et al. (2013), that the FRP wrap provided considerable additional strength at all temperatures tested, particularly for the case with three continuous layers of carbon fibre reinforcement. For the case of three layers of FRP, the confinement continued to enhance the failure strength at temperatures well above \( T_g \). For example, strength was enhanced by more than 200% at 400°C. The confinement provided by the single layer of FRP is much less; this is likely the result of frictional bond strength for the three layer wrapping, which is able to provide confinement even once the adhesive has lost the majority of its mechanical properties.

Tests on TRM Confined Concrete

For the TRM confined concrete cylinders, Figure 1 and Table 2 show that the TRM system was marginally less effective than the FRP system for a single layer at ambient conditions, and that it’s performance was similarly affected by exposure to a temperature of 100°C, losing 23% of its strength at this temperature. This is likely due to a combination of reductions in the strength of the concrete itself at this temperature (discussed previously) combined with reductions in the effectiveness of the confinement (for reasons which remain unknown). However, at temperatures above 100°C the TRM system recovered its strength more rapidly than either the plain or FRP confined specimens, and at 400°C the single layer TRM confined sample actually tested stronger than the sample tested at ambient.

Similar observations can be made regarding the performance of the three layer TRM confining system. In this case the strengthening provided by the TRM system was considerably less, by 33%, than the three layer FRP system, probably because of differences in the observed failure mode; tensile pull-out of the fibres from the cementitious matrix followed by partial fibre fracture in the case of the TRM, compared with tensile fibre fracture over the height of the cylinder in the case of the FRP. Figure of the TRM wrapped specimens was considerably less violent and absorbed more energy than for the FRP wrapped specimens, particularly at lower temperatures. This may present advantages, particularly in seismic strengthening applications. However, at elevated temperature the three layer TRM system displayed similar response as for the single layer TRM system, with mild reductions in strength.

![Figure 19](image-url)

Figure 19 Ultimate Compressive Strength and Percentage Strengthening versus Temperature
at 100, 150, and 200°C (likely for the reasons discussed in the previous paragraph), but a small increase (by 7%) in strength at 400°C as compared with the strength at ambient.

Increases in the effectiveness of the TRM system at elevated temperature are striking, particularly for the case of the three layer TRM system, and may be due to thermal prestressing of the carbon fibres during heating, as has previously been suggested by Rickard et al. (2013) for multiple layer FRP confining systems at elevated temperature. Since carbon fibres generally have a negative coefficient of thermal expansion, whereas the concrete core has a positive coefficient of thermal expansion, thermal dilation of the concrete core during heating will develop tensile stresses in the carbon fibre mesh, and provided that bond is maintained (either by adhesion or by winding friction) the wrap will self-press, which can be expected the enhance both the strength and stiffness of the TRM confined cylinders. This hypothesis warrants additional research, and may indicate that unprotected TRM systems are capable of providing effective (i.e. active) confinement of concrete at temperatures up to and exceeding 400°C (something that epoxy-based FRP systems are unable to do).

**Axial/Hoop Stress versus Strain Response**

Typical stress-strain curves obtained during steady state testing using the image correlation analysis are given in Figure 4. For clarity only the ambient temperature, specimens confined with three layers, 150°C, and 400°C tests are shown. It was difficult to obtain consistent strain information during these tests due to thermal effects on the image correlation technique; the results should therefore be taken as only indicative of the response. It is noteworthy that both the strength and stiffness of the confined concrete appear to be significantly reduced at 150°C, whereas these appear to be recovered at 400°C. Again, this behaviour warrants further investigation, both for confinement research but also for structural fire engineer research on concrete structures. Further image correlation analysis results will be presented at the conference.

![Figure 20 Stress-Strain Curves for Cylinders with Three Layers of Strengthening: (a) FRP wraps, and (b) TRM wraps](image)

**PRELIMINARY CONCLUSIONS**

The following preliminary conclusions can be drawn on the basis of the tests reported herein:

Considerable loss of effectiveness of the FRP wrap system occurred at temperatures as exceeding the $T_g$ of the epoxy adhesive used. This is thought to be due to reductions in the tensile, and more importantly bond, strength of the FRP wraps at these temperatures.

The ultimate load capacity of FRP wrapped cylinders continued to decrease with increasing temperatures; however the FRP wrap continued to provide some confinement at all exposure temperatures, both for a single layer and for a triple layer, even at 400°C which is well above $T_g$. This is possibly attributed to bond retention resulting from winding friction, which appears to be (as expected), much more effective for three continuous layers of FRP than for a single layer.
FRP rupture was the observed failure mode at temperatures below 100°C; above 100°C the failure mode was observed to transition to bond failure in the FRP overlapping zone.

Minor loss of effectiveness of the TRM wrap system occurred at temperatures between 100 and 200°C. This is thought to be due to reductions in the compressive strength of the concrete at these temperatures (as corroborated by compression tests on unconfined concrete at these temperatures). However, both single and triple layer TRM wrap systems demonstrated enhanced strength at 400°C.

It appears that differential thermal expansion between the carbon fibre reinforced wraps (contracting) and the concrete column (expanding) caused a slight prestress of the FRP and TRM systems during heating.

The number of FRP and TRM layers of wrapping (hence the effective overlap length used) had a significant impact on the confinement effectiveness at elevated temperature; however, additional testing is required to study the influence of wrapping with multiple continuous layers on the performance of FRP and TRM confined concrete at elevated temperature.

**RECOMMENDATIONS**

The following are recommendations for future work in this area:

Further tests on unconfined concrete at elevated temperature are required to better understand the effects of heating on reduction in unconfined concrete compressive strength.

Additional repeat tests, particularly on FRP and TRM confined concrete, are required to ensure statistical confidence in the results obtained and to corroborate the results and hypotheses presented herein.

Other available FRP and TRM wrap systems and materials need to be studied to allow generalization of the observations made during the current study.

Cylinders tested in this project were subjected to loads at elevated temperatures for relatively short periods of time. The potential effects of prolonged loading and heating, as might occur in warm service temperatures, require study since creep may cause failure at lower temperatures under long term loading.

Additional tests should be conducted on FRP and TRM confined RC columns at full-scale.

**ACKNOWLEDGEMENTS**

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DIAGONAL SHEAR TESTS OF FULL SCALE FRM-STRENGTHENED LIMESTONE MASONRY WALLS

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ABSTRACT

Fiber Reinforced Polymer (FRP) materials have been extensively studied and applied all over the world in the last two decades. Nowadays a new generation of inorganic composite materials is also widely studied for seismic strengthening of ancient masonry construction. The new technique is studied for those cases in which a localized epoxy-bonded reinforced show its limits due to the weak nature of the substrate. In these cases Fiber Reinforced Mortar (FRM) systems are used as external strengthening coating, in forms of fibrous grids immersed in a brittle matrix made by hydraulic or cementitious mortar. The results of an extensive experimental program are presented in this study, which involved twelve full scale masonry walls subjected to diagonal shear tests. Two different configuration of masonry walls were studied: six walls were built with a single layer of limestone masonry blocks and six walls were built with a double curtain of limestone blocks separated by a layer of inconsistent mortar and connected by six transverse bond-stones. Low-strength hydraulic mortar was used as binder between the blocks. These configuration were chosen to replicate typical walls found in real heritage masonry buildings. Two different strengthening FRM systems were applied on both sides of the specimens and studied. The first consisted of an industrially pre-cured rigid Glass FRP grid immersed in a lime-based mortar. The second system was an hybrid system made by a dry glass fiber net immersed in a lime-based mortar in which a polymer primer was added as bond promoter. The results of the mechanical tests highlighted in all cases a remarkable increase of shear strength due to the weak properties of the unstrengthened walls. Brittle failure due to diagonal sliding along the joint was observed in unstrengthened masonry. The presence of the FRM has resulted in a pseudo-ductile behaviour of the specimens in both cases, even if different failure modes were observed for the two FRM systems. Fiber rupture and a full composite action was observed for the polymer added FRM system, while extensive cracks and local spalling was observed by testing the fully inorganic system.

KEYWORDS

FRM, FRP, masonry, shear, strengthening, composites.

INTRODUCTION

Structural deficiencies of poor masonry buildings under horizontal forces are well known, and protection of the cultural heritage has become a priority when seismic hazard is considered. Due to the low mechanical properties of masonry constituents, and lack of proper design, horizontal forces may cause shear failures in loading bearing walls which lose their stiffness and strength under cyclic actions. Seismic events all over Europe and Asia, in the last decades, had consequences that have alerted scientists and engineers, due to the brittle failures of masonry buildings. When severe earthquakes forces invest a masonry system two scenarios may open: 1) the masonry is not connected and thus local disaggregation or local rigid overturning cause collapse; 2) masonry is well connected and internal forces are carried by shear walls. In this second case the seismic vulnerability is related to the shear strength of the masonry panels, which often incorporate poor materials such as soft stone and weak limestone mortars. A quantitative study on how the weak hydraulic masonry is able to carry shear forces was conducted by Augenti & Parisi (2011). When the mortar is poor, failure is due to sliding along the blocks; when the mortar is strong, shear cracks pass through the blocks. In the last decade innovative techniques were studied and applied in order to increase the in-plane shear strength of masonry walls. Fiber Reinforced Polymer (FRP) materials were investigated in forms of laminates (Valluzzi \textit{et al.} 2002). The effectiveness of FRP-strengthening was confirmed in recent studies also for historical limestone masonry (Marcari \textit{et al.} 2011) and bricks (Saghafi \textit{et al.} 2013). Recently new materials with inorganic matrix were developed and studied for the strengthening of ancient masonry (Carozzi \textit{et al.} 2014), in these cases the bonding agent is not a polymeric resin and its application is diffused and not localized. Inorganic matrixes, mostly cement based, were proposed in recent studies as strengthening systems.
with a continuous fibrous reinforcement (Babeidarabad 2014; Gattesco & Dudine 2010, Gattesco et al. 2015; Menna et al. 2015; Yardim et al. 2016). In this experimental study twelve full scale masonry walls were built and tested in diagonal shear until failure. Limestone masonry blocks were regularly packed by using a weak limestone mortar, to simulate historical construction found all over southern Europe. Two type of specimens were tested: single wall and double head-faced walls. The results show that the low shear properties of the tested masonry dramatically increased after the installation of the FRM reinforcements.

**EXPERIMENTAL PROGRAM**

Twelve walls specimens were built with two different construction schemes, to simulate the real findings in historical buildings. Six were built as single walls (“S2” labels), the remaining six specimens were built as double walls (“D2” labels). In this case the two masonry curtains were separated by a non structural thickness of 80 mm made by inconsistent mortar. They were connected by six transverse bond-stones as shown in Figure 1, in which the design geometry of the specimens is illustrated. The geometry of the specimens was studied in order to maintain a slenderness ratio of 12 between the height and the structural thickness. Therefore in doubled wall specimens the structural thickness is the same to that of single walls.

![Figure 1 Geometry of wall specimens (quotes in mm)](image)

The mortar that was used to build the specimens was a poor limestone-based mortar without cementitious binder, as same as it was done by the ancient master builders. Mortar specimens were poured and stored under the same laboratory conditions of the wall specimens. The mechanical properties of the mortar were measured in laboratory by testing 10 specimens for each type of test. A compressive strength of 1.68 MPa, an elastic modulus of 4478 MPa, a tensile strength of 0.13 MPa, and a mass density of 1592 kg/m$^3$ were found. The experimental program, specimen dimensions and type of FRM reinforcement are reported in Tables 1 and 2.

A mechanical characterization of the masonry was also carried out. Masonry specimens were tested under compression to measure the nominal strength of the material. Three single wall specimens (1000x1200x160 mm) and three double-face walls (800x650x150 mm) were subjected to compression load until crushing failure. The average strength of the single walls was 4.38 MPa, while for double-face the average strength was 6.01 MPa. Three double face-walls were also tested by applying the load at 90° respect to the masonry stacking, in direction parallel to the horizontal joints; in this case the average ultimate strength was 4.12 MPa.

Two different strengthening systems were used for the shear reinforcement that was applied on both sides of the specimens. The first system is represented by a rigid Glass FRP (GFRP) grid embedded in a coating which is made by an hydraulic lime mortar. This technique is an evolution of the traditional steel-reinforced coatings, but with advantages of reduced thickness and absence of galvanic corrosion. The GFRP grid was industrially formed by using AR-glass (alkali resistant) fibres and an epoxy/vinylester resin, and it has a 66x66 mm spacing. Four GFRP mechanical anchors were provided in forms of bent pultruded bars injected across the masonry thickness, as shown in the representative Figure 2. This system can be applied by using traditional construction techniques. The mechanical properties of the GFRP grid were provided by the manufacturer with reference to the composite section: tensile strength 1000 MPa, ultimate strain 3%, elastic modulus 27 GPa.

The second reinforcement system is represented by a dry AR glass mesh having a density of 220 g/m$^2$ immersed in a limestone-based matrix which is applied to the masonry by using a polymer-added primer that is able to generate crosslinking in presence of water. The primer has the double function of bond promoter between masonry and mortar, and between mortar and fibers. Four FRP connectors in forms of CFRP spike-bars were applied. The procedure does not require temperature or moisture control as done in presence of epoxy resins as mandatory.
advantage of this system is represented by the high bond retention (similar to FRP), very low thickness (less than 10 mm) and ease of installation. The strength and elastic modulus of the glass fibers are respectively 1500 MPa and 70 GPa as reported by the manufacturer. Two repetitions for each unstrengthened and strengthened schemes were tested by using a test set-up illustrated in Figure 2. The test set-up has a steel angle-toe welded to a stiffened H-shape profile, to avoid deformation of the loaded angle. A second similar device was applied at the opposite corner of the specimen. Finally, a third device was connected to the bottom device. Four steel bars were used as diagonal tendons, and the diagonal force was provided by a hydraulic jack (maximum axial load equal to 933 kN, stroke of 257 mm) interposed between the top devices. Four LVDTs were placed along the four diagonals of the specimens (both sides), and two additional horizontal LVDTs were placed on both sides at the centre, to detect potential out-of-plane displacements. The reference points on the diagonals were placed at a distance of 1050 mm. Diagonal compression load and signals from displacement transducers were contemporarily recorded by an electronic data logger.

![Shear reinforcement scheme with GFRP reinforced coating - Experimental diagonal shear set-up](image)

For all specimens three load-unload cycles were run at low values of the applied load to press the voids that are present inside the masonry system. This allowed to avoid strain readings that are not relevant with the real mechanical behaviour of the panel. In Tables 1 and 2 the experimental program is resumed.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>FRM reinforcement system</th>
<th>Real dimensions BxH (mm)</th>
<th>Total thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2-NR-1</td>
<td>Unreinforced</td>
<td>1970x1990</td>
<td>160</td>
</tr>
<tr>
<td>S2-NR-2</td>
<td>Unreinforced</td>
<td>1960x2010</td>
<td>160</td>
</tr>
<tr>
<td>S2-FB8-1</td>
<td>Rigid GFRP mesh 66x66 mm</td>
<td>1970x2010</td>
<td>200</td>
</tr>
<tr>
<td>S2-FB8-2</td>
<td>Lime mortar 8 MPa with GFRP anchors</td>
<td>1970x2000</td>
<td>200</td>
</tr>
<tr>
<td>S1-FRCM-A1</td>
<td>Dry glass fibers net Polymer added Lime</td>
<td>1960x2000</td>
<td>170</td>
</tr>
<tr>
<td>S1-FRCM-A2</td>
<td>mortar with CFRP anchors</td>
<td>1960x1990</td>
<td>170</td>
</tr>
</tbody>
</table>

B= width of the wall; H=height of the wall.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>FRM reinforcement system</th>
<th>Real dimensions BxH (mm)</th>
<th>Total thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D2-NR-1</td>
<td>Unreinforced</td>
<td>1970x2020</td>
<td>250</td>
</tr>
<tr>
<td>D2-NR-2</td>
<td>Unreinforced</td>
<td>1970x2020</td>
<td>260</td>
</tr>
<tr>
<td>D2-FB8-1</td>
<td>Rigid GFRP mesh 66x66 mm</td>
<td>1980x2030</td>
<td>290</td>
</tr>
<tr>
<td>D2-FB8-2</td>
<td>Lime mortar 8 MPa with GFRP anchors</td>
<td>1980x2020</td>
<td>300</td>
</tr>
<tr>
<td>D1-FRCM-A1</td>
<td>Dry glass fibers net Polymer added Lime</td>
<td>1920x2010</td>
<td>270</td>
</tr>
<tr>
<td>D1-FRCM-A2</td>
<td>mortar with CFRP anchors</td>
<td>1980x2010</td>
<td>270</td>
</tr>
</tbody>
</table>

B= width of the wall; H=height of the wall.
RESULTS AND DISCUSSION

Experimental results highlighted the mechanical benefit that was provided by the presence of the FRM reinforcement systems, even if important differences may be remarked. Different failure modes were found since the strengthening system were very different. In Figure 3 they are illustrated, and in Tables 3 and 4 a synthesis of the experimental results is listed. It can be seen that the maximum shear load carried by the walls increased up to 8 times for single walls and up to 5 times for double walls. In both cases unreinforced specimens had a typical failure with sliding along the mortar joints. This occurrence testified the important role of the connection produced by the transverse blocks. After the crack opening along the compression strut the specimens did not show a significant residual capacity. This brittle failure mode is typical of masonry panels subjected under in-plane loads when the strength of the mortar joint is much lower than the strength of the blocks. Crushing phenomena in the blocks were not evident, while a macroscopic misalignment of the joints was visible at the end of the test. Specimens strengthened with rigid GFRP grid in limestone coating exhibited a progressive cracking opening, which started on the compressed diagonal and then spread along the entire panel.

![Figure 3 Failure modes of the tested specimens](image)

Table 3 Experimental results single wall specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>P&lt;sub&gt;M&lt;/sub&gt; Maximum Load (kN)</th>
<th>P&lt;sub&gt;AV&lt;/sub&gt; Average (kN)</th>
<th>P&lt;sub&gt;M&lt;/sub&gt;/P&lt;sub&gt;MU&lt;/sub&gt;</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2-NR-1</td>
<td>30.3</td>
<td>31.3</td>
<td>-</td>
<td>Diagonal sliding joints</td>
</tr>
<tr>
<td>S2-NR-2</td>
<td>32.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S2-FB8-1</td>
<td>210.7</td>
<td>215.1</td>
<td>6.9</td>
<td>Extensive diagonal cracking and local spalling of the coating</td>
</tr>
<tr>
<td>S2-FB8-2</td>
<td>219.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S2-FRCM-A1</td>
<td>237.6</td>
<td>244.0</td>
<td>7.8</td>
<td>Horizontal sliding with fibers slip and tensile rupture</td>
</tr>
<tr>
<td>S2-FRCM-A2</td>
<td>250.3</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In double-face walls at high loads failure was accompanied also by opening of the free corner, even if this was not the direct cause of the collapse. Local spalling of the coating was observed and even under severe damage the wall was able to maintain its load capacity. In the case of the second FRM system, in which a polymeric primer was used, debonding of the coating was not observed, and cracks were mostly horizontally directed. Failure was
anticipated by fibres slippage inside the matrix, and was due to fibre rupture accompanied by horizontal sliding of the wall. After fibre rupture, spalling of the inorganic skin was observed.

In Figure 4 (a&b) the load versus diagonal strain curves are reported for all the tested specimens; with reference to an average value obtained from both sides of the walls along the compressed diagonal. All the averaged curves that are shown in the paper are stopped in correspondence of a strain equal to 0.003 since in many cases, after this point the comparison may be affected by local effects.

<table>
<thead>
<tr>
<th>Table 4 Experimental results double wall specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen</td>
</tr>
<tr>
<td>----------</td>
</tr>
<tr>
<td>D2-NR-1</td>
</tr>
<tr>
<td>D2-NR-2</td>
</tr>
<tr>
<td>D2-FB8-1</td>
</tr>
<tr>
<td>D2-FB8-2</td>
</tr>
<tr>
<td>D2-FRCM-A1</td>
</tr>
<tr>
<td>D2-FRCM-A2</td>
</tr>
</tbody>
</table>

Extensive damage of the specimens, or local crushing at the corners in fact produced a strong loss of stiffness leading to different softening branches. Even if in all cases the presence of the FRM produced a huge increase in strength it is important to distinguish the different behaviour of the curves after the reverse point, due to the different strengthening systems. In the case of GFRP reinforced coating the plateau seems horizontal, with a light decreasing trend, due to cracking. While in the case of polymer-added glass-FRM system an hardening trend is visible, due to the stress that arises in the glass fibres. Local sudden loss of load were due to loose of friction and local slipping, which was not the cause of the failure, since the ultimate strain of the fibres was exploited along the central region of the panels.

Figure 4 Load vs diagonal compression strain experimental diagrams

The most important result that was evident after the tests was the change of the masonry in terms of mechanical response. The unreinforced masonry, that showed brittle failure after the formation of the diagonal strut, turned into a new reinforced state, when FRM was installed. In fact after the failure of FRM specimens, the inspection of the masonry core revealed a monolithic masonry panel under the coating, and its stiffness was maintained by the presence of the fibres also for loads that were four five times the ultimate of unreinforced masonry. Another important aspect to be discussed is that even if differences in terms of failure mode and maximum loads were recorded with reference to the different FRM systems, in all cases the failure was not due neither to delamination of the reinforcement layers, nor to the debonding of the FRP anchors. This indicates that stress transfer mechanisms are different if compared to the case of FRP-strengthened walls, in which the ultimate capacity is entrusted to the local bond stresses between the masonry substrate and the FRP strips. The pure composite action, that is desirable in FRP-strengthened substrates, was not found in walls with GFRP reinforced coating and was present only at the
first loading stages in walls reinforced with the second glass-FRM system, afterward the stress transfer was due to friction between fibres and matrix until fiber rupture.

CONCLUSIONS

An experimental program was carried out in order to investigate the effectiveness of two different FRM systems in shear strengthening of masonry walls made with regular limestone blocks and weak hydraulic mortar. Two different construction techniques, widely found in the field were considered: single walls and double curtain walls, with transverse blocks used as connectors. Different fibrous strengthening systems were used, a rigid GFRP grid and a soft dry glass fiber net. The first was immersed in a 8 MPa hydraulic lime mortar, the latter was immersed in a limestone mortar added by a polymeric primer which is able to increase the cross linking in presence of water. After a complete mechanical characterization of the constituent materials, shear diagonal tests were performed on unreinforced and FRM-reinforced masonry. Totally twelve samples were tested until failure. Unreinforced masonry, as expected, exhibited a brittle failure at low load levels, due to the sliding along the mortar joints of the diagonal struts. Important remarks may be reported by considering the results obtained in all cases, which can be considered as a first milestone for future studies:

the presence of the FRM reinforcing systems strongly increased the load capacity of the plain masonry. Both in the cases of single or double walls, the peak load exceeded from 5 up to 8 times the capacity of the unreinforced masonry;

the effectiveness of the FRM reinforcement was higher in single walls respect to double curtain walls;

the presence of the FRM reinforcing systems changed the mechanical behaviour of the masonry in terms of failure modes and strain energy retention;

the presence of the FRM reinforcement avoided the complete disaggregation of the masonry panels, which in the real field is the main cause of losses in human lives;

the failure modes were not caused by delamination or debonding of the strengthening FRM systems, this recalls the attention about the importance of transverse connections made by four injected FRP bars per each specimen.

Thus the behaviour of FRM-strengthened masonry appears far from the failure modes found in FRP-strengthened masonry.

Extensive cracking was visible in FRM-strengthened masonry, especially when pure limestone mortar was used as matrix. In this case the diagonal strut region appeared to be enlarged due to the new stress field developed by the presence of the internal fibrous reinforcement. Friction and slippage between fiber and matrix anticipated fiber rupture and was observed in the polymer added glass FRM system. Due to the low number of experimental data, further results will be necessary in order validate or discuss the conclusions of the present study.

ACKNOWLEDGMENTS

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DURABILITY PERFORMANCE OF FRCM COMPOSITE BONDED TO CONCRETE UNDER DIFFERENT ENVIRONMENTAL AGING CONDITIONS

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ABSTRACT

From the perspective that an aggressive environment may cause damage to cementitious materials, curing agents of fiber reinforced cementitious matrix (FRCM) composite allows chemical agents to attack the reinforcement fabrics. In such cases, the accelerated interfacial FRCM debonding mechanisms interfere which reduce its mechanical performance. This task was conducted to investigate the long-term durability performance of the FRCM composite. In this study, environmental aging conditions were the freeze-thaw cycles, high humidity cycles, high temperature cycles, immersion into salt solution, and immersion into alkaline solution. Different FRCM reinforcement ratio and two surface roughness of concrete were also included. Two test methods were used to evaluate the FRCM composite’s bond performance (pull-off test and bending test). The bending test results revealed that the FRCM composite bond performance was not influenced by the environmental exposure. While the test results for the pull-off strength were scattered, possibly due to the environmental degradation or a lack of proper quality control during the initial FRCM composite application or applying the load.

KEYWORDS

FRCM composite, bond, pull-off test, bending test, environmental conditioning.

INTRODUCTION

Using fiber reinforced cementitious matrix (FRCM) to repair deteriorated, damaged, or structurally unsafe concrete members, is an increasingly popular technique. The previous experimental works have been proving that FRCM composite application is economic, durable, convenient, and labor friendly (Babaeidarabad et al. 2014; Ombres 2011; Loreto et al. 2014). The long-term in-situ performance of the FRCM composite-concrete interfacial bonding is a primary concern due to the potential for age-related environmental degradation. A study on the bond strength-slip relations for the PBO-FRCM composite externally bonded to concrete blocks was reported by D’Ambrisi et al. (2012). Carloni et al. (2013) conducted an experimental study on the applicability of a fracture mechanic based approach to understand the stress transfer mechanism of FRCM composites externally bonded to a concrete substrate. Results were analyzed to determine the effective bond length, which can be used to establish the load-carrying capacity of the interface to design the strengthening system. Results also determined the interfacial behavior between fibers and matrix and highlight the role of the cementitious matrix in the stress transfer. D’Ambrisi et al. (2013) experimentally analyzed the bond between FRCM composite made out of a poly(paraphenylene benzobisoxazole) (PBO) net embedded in a cement based matrix and the concrete. The results allowed estimating the effective anchorage length and evidence that the debonding occurs at the fibers/matrix interface after a considerable fibers/matrix slip. The results also confirm the effectiveness of the FRCM materials as external reinforcements for concrete and the obtained experimental results can be used to calibrate a local bond-slip relation in the design of the external reinforcement. One other study related to D’Ambrisi et al. (2013) is an experimental and analytical investigation on bond between carbon-FRCM composite and masonry. Experimental results of double shear tests involving different bond lengths can be used to calibrate a local bond-slip relation that is essential in the modeling of the structural behavior of masonry elements strengthened with carbon-FRCM. D’Antino et al. (2014) conducted a single-lap shear test on specimens with FRCM composite strips bonded to concrete blocks. Test parameters included different FRCM composite attachment area and reinforcement ratio and the stress-transfer mechanism that characterized the FRCM composite was determined. Arboleda et al. (2014) studied the durability of FRCM composite in terms of different environmental exposure under bond pull-off testing. Two types of FRCM composites were investigated (carbon fabric and PBO fabric with cement based matrix).
results for tensile and bond strength retention percentages after environmental exposure revealed no significant degradation concerns. Sneed et al. (2015) demonstrated a comparison study between single-lap and double-lap shear test on the bond failure mechanism of PBO-FRCM composite. Ombres (2015) shaded the light on a comparison between experimental results and theoretical predictions of the bond-slip law for PBO-FRCM composite attached to concrete. The comparison results concluded that the nonlinear proposed model described analytically the bond-slip law is influenced by experimental results and the available experimental data are not sufficient for a reliable calibration of parameters that define the model. This experimental work was looked up other aspect of view to investigate the long-term durability performance of bonded FRCM composite to concrete substrate. The environmental aging conditions were the freeze-thaw cycles, high humidity cycles, high temperature cycles, immersion into salt solution, and immersion into alkaline solution that were based on the recommendations of AC434 (2013).

EXPERIMENTAL PROGRAM

Description of FRCM composite

The FRCM composite that considered in this study was consisted of poly(paraphenylene benzobisoxazole) (PBO) fabric and cement based matrix. The PBO fabric was made of spaced strands in two directions. The PBO fabric strand has a width of 5 mm (0.2-in.) in the longitudinal direction and a width of 3 mm (0.12-in.) in the transverse direction. The free spacing between the strands is approximately 5 mm (0.2-in.) and 22 mm (0.9-in.) in the longitudinal and transverse directions, respectively. The nominal thickness of the strands is 0.2 mm (0.008-in.) and 0.12 mm (0.0045-in.) in the longitudinal and transverse directions, respectively. As reported by the manufacturer, the PBO fabric’s ultimate tensile strength per unit width is 276 kN/m (18.9 kip/ft) in the longitudinal direction which is about 3.5 times its’ tensile strength in the transverse direction. The bonding agent is a cement based mortar that has less than 5 percent polymer. Figure 1 shows the PBO-fabric and the cementitious matrix.

![Figure 1 a) Polyparaphenylene Benzobisoxazole (PBO) mesh, b) Inorganic Matrix](image)

Specimen preparations

Two test methods were used to evaluate the FRCM composite’s bond performance (pull-off test and bending test). The test matrix was divided into two phases of specimen preparations. In the first phase, the specimens were prepared for pull-off test. In the second phase, the specimens were prepared for bending test.

Specimen preparation and conditioning for pull-off test

In this phase, concrete blocks with a 508 mm (20-in.) length, a 150 mm (6-in.) width, and 150 mm (6-in.) height were casted for attaching one layer of FRCM composite. The average compressive strength of the three tested cylinders was 41 MPa (6000 psi) at 28 days based on ASTM C39 (2014). All of concrete substrate surfaces were roughened in order to provide better bond performance with FRCM composite. The sandblasting method was used for concrete surface roughness to a level of 0.5 mm (0.02-in.). That penetration level simulated the surface profile CSP3 when the concrete specimens were lightly sandblasted (Technical guidelines No.03732 and No.03730, International Concrete Repair Institute (ICRI), 1997). For FRCM composite installation, the hand lay-out method was followed with respect to the guidelines of ACI 549 (2013). The installation method consisted of four steps. The first step was to wet the concrete surface. The second step was to apply the first layer of the cementitious mortar (X MORTAR 750) with a nominal typical thickness of 3 mm (0.1-in.). The third step was to place the PBO-fabric and press it gently into the cementitious mortar. The fourth step was to apply the second layer of the cementitious mortar to the same thickness, and level FRCM composite surface. All of the concrete specimens cured for 28 days under laboratory conditions before any exposure or testing. The environmental exposure and aging were the main parameters for this test. The exposure conditions and aging were based on the recommendations in the AC434 (2013). The specimen’s matrix for pull-off test included five exposure conditions, as represented in Table 1. In the first conditioning, specimens were mentioned under laboratory conditions that
served as control specimens. In the second conditioning, specimens were exposed to an environmental regime that included 50 cycles of freezing and thawing, 150 cycles of humidity, and 150 cycles of high temperature. In the third conditioning, specimens were placed inside a moisture chamber under 100% relative humidity. In the fourth conditioning, specimens were immersed into a salt solution that demonstrated the case of exposure to seawater (PH level of 7). In the fifth conditioning, specimens were immersed into an alkaline solution that demonstrated the case of ocean water (PH level of 13). The aging condition was the third study parameter. Some of the specimens were tested after one environmental regime cycle (72 days) and other specimens were tested after an aging time of 166 days based on the recommendation of AC434 (2013). The environmental regime was established according to Missouri state weather conditions. The data was collected from the National Weather Service and Worldwide Weather Station during a time frame from 1980 to 2013. The environmental regime is presented in Figure 2. In the test matrix, the defined specimen name is represented by two letters and one number. The first letter, P, represents the test type (pull-off). The second letter, L, E, H, S, or A, represents the exposure condition (laboratory, environmental, humidity, saltwater, or alkaline solution). Finally, the number, 1, represents the number of ply. The salt solution was prepared by mixing NaCl of 3% by the weight of required solution to immerse the specimens. While, the alkaline solution was prepared by adding NaOH of 0.4% by the weight of the required solution. 

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Tested specimen number</th>
<th>Conditions</th>
<th>Plies number</th>
<th>Exposure time, days</th>
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</thead>
<tbody>
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<td>1</td>
<td></td>
</tr>
<tr>
<td>P-E-1</td>
<td>5</td>
<td>Environmental regime cycles</td>
<td>1</td>
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</tr>
<tr>
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<td>100% humidity at 72 °F</td>
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<td>72</td>
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<tr>
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<td>1</td>
<td>72 166</td>
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<tr>
<td>P-A-1</td>
<td>5</td>
<td>Alkaline solution at 72 °F</td>
<td>1</td>
<td>72 166</td>
</tr>
</tbody>
</table>

Figure 2 Environmental conditioning regime

Specimen preparation and conditioning for bending test

One test specimen consisting of two concrete blocks were spaced 12.5 mm (0.5-in.) by wood piece and a strip of the FRCM composite was attached to one face of the connected concrete blocks. The concrete blocks were casted to have a 254 mm (10-in.) length, a 150 mm (6-in.) width, and a 150 mm (6-in.) height. The average compressive strength of the three tested cylinders was 41 MPa (6000 psi) at 28 days based on the ASTM C39 (2014). Two different surface preparations were considered in order to examine the effect of concrete surface roughness on FRCM composite bond performance. The first roughness level of 0.5 mm (0.02-in.) simulated the surface profile CSP3 when the concrete specimens were lightly sandblasted. The second roughness level of 1 mm (0.04-in.)
simulated the surface profile CSP6 or CSP7 when the concrete specimens were medium-heavy sandblasted. These penetration levels were based on technical guidelines No.03732 and No.03730 specified by International Concrete Repair Institute (ICRI, 1997). The FRM composite strip had a 100 mm (4-in.) width and 432 mm (17-in.) length. The FRM composite installation procedure and curing were as preceded for the pull-off test specimens, except that the installation procedure was repeated successively for the specimens with four plies of FRM composite. The specimens’ matrix for bending test included two exposure conditions (laboratory condition and environmental regime cycles), as represented in Table 2. The environmental regime cycles are represented in Figure 2. In the test matrix, the defined specimen name is represented by three letters and one number. The first letter, B, represents the test type (bending). The second letter, L or E, represents the exposure condition (laboratory or environmental). The third letter, L or H, represents the surface roughness (light or heavy). Finally the number, 1 or 4, represents the number of plies.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Specimen tested number</th>
<th>Conditions</th>
<th>Surface Roughness</th>
<th># of plies</th>
<th>Exposure time days</th>
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<td>1</td>
<td>72</td>
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<tr>
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<tr>
<td>B-E-H-4</td>
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<td>Environmental regime cycles</td>
<td>Heavy</td>
<td>4</td>
<td>72</td>
</tr>
</tbody>
</table>

**TEST METHODS**

**Pull-off test**

This test method was implemented in order to examine various environmental exposures on the bond performance of FRM composite. The pull-off test was performed based on the general procedure of the ASTM C1583 (2013). A minimum of five specimens for each FRM composite exposure were tested as recommended by ACI 549 (2013). After specimens were aged in exposure conditions, they were left in laboratory conditions for seven days to dry out, and then a 50 mm (2-in.) diameter bit was used to perform 12.5 mm (0.5-in.) depth cores. A vacuum cleaner was used to get rid of all the concrete dust then steel disks attached to FRM composite surfaces by epoxy adhesive. The specimens were left to cure the epoxy adhesive for at least three days before testing. The pull-off tester was used to pull the attached steel disks to the cored specimens. The incremental load was applied to produce an approximate stress rate of 35 kPa/second (5 psi/second). Figure 3a shows a representative specimen under pull-off testing. The pull-off test revealed the pull-off strength and the bond failure mode.

![Figure 3 a) Specimen under pull-off test, b) Specimen under bending test](image)

**Bending test**

This test method was selected to reveal the impact of surface roughness and environmental regime cycles on multilayers of FRM composite. The test setup was designed to simulate a reinforced concrete beam in bending with a preexisting crack at the mid-span (Pellegrino et al. 2008; Silva et al. 2008). All of the specimens were loaded at two points that spaced 150 mm (6-in.) apart and simply supported at each end thereby subjecting the FRM composite to bending stresses.
composite to a direct tension. Two linear variable differential transducers (LVDT) were used to measure the mid-span displacements. All of the specimens were subjected to a constant load rate of 0.25 mm/minute (0.01 in/minute). A representative specimen under a bending test is seen in Figure 3b. The bending test results revealed the FRCM composite’s tensile strength, mid-span displacements, and failure mechanism.

RESULTS AND DISCUSSIONS

Pull-off test results

The pull-off strength of five specimens based on the exposure condition is represented in Figure 4a. The pull-off strength was computed by dividing the peak load over the net cross-sectional area that specified by ACI 549 (2013). Bond failure was observed in all specimens at the interface between the PBO fabric and the cementitious matrix, as seen in Figure 4b. All of the specimens had pull-off strength greater than the 1.4 MPa (200-psi) defined as a minimum value by ACI 549 (2013). The test results had a large scatter for all types of exposure, which might be due to the manual control in applying the load. The specimens exposed to the salt solution and alkaline solution had lower pull-off strength in comparison with other exposure conditions at 72 days aging, while the aged specimens to 166 days had higher pull-off strength. That concluded the non-influential effect of using such composite in repairing or strengthening applications that exposed to salt or alkaline environment.

Bending test results

The bending test results were determined as the average tensile strength of three tested specimens, which is represented in Figure 5a. The tensile strength was calculated from Eq. (1):

$$\sigma = \frac{P}{nb}$$

(1)

where, \(\sigma\) is the tensile strength, MPa (Psi), \(P\) is ultimate load, \((N \text{ or } Lb)\), \(n\) is the number of PBO fabric plies, \(b\) is the width of FRCM composite strip, and \(t\) is the thickness of FRCM composite strip, which was equal to 5 mm (0.2-in.) for one ply and 10 mm (0.4-in.) for four plies. The tensile strength results revealed that the number of FRCM composite plies was the most influential factor on the failure mode type and tensile strength of the tested specimens. The performance of the specimens with a heavily roughened surface was insignificant in comparison to the specimens with lightly roughened surface. In addition, the environmental regime cycles showed insignificant effect on the tensile strength of the tested specimens. The strengthened concrete blocks with one ply of FRCM composite exhibited a slippage bond failure mode, as seen in Figure 5b, while the strengthened concrete blocks with four plies of FRCM composite failed due to the FRCM composite’s debonding from the concrete substrate, as seen in Figure 5c. The representative applied loads with specimens’ displacements at the min-span are seen in Figure 6a and 6b, respectively. The global slip is defined as a specimen’s displacement at the mid-span where a maximum slippage occurred in the PBO fabric after increasing the applied load. The applied load-global slip relationships were influenced by the number of FRCM composite’ plies used. For concrete blocks with one ply of FRCM composite, the load started to increase linearly up to the point where slippage in the PBO fabric started to enlarge. Then, the load was increased to ultimate values with observed loss in its stiffness. As the specimen reached its’ ultimate load capacity, the load capacity was incrementally dropped to the point where the global slip was continuously extended at a constant load. This phenomenon was observed in single lap shear test that was conducted in previous research (D’Antino, 2014). The shear stress due to the friction force between the PBO fabric and the bonding agent (cementitious mortar) revealed the continuous global slip (D’Antino, 2014). However, for concrete blocks with four plies of FRCM composite, the specimens exhibited a rapid linear drop in their load capacities when the FRCM composite debonded from the concrete substrate, as seen in Figure 6b. The bending test was able to predict the failure mode with respect to the FRCM composite’s reinforcement ratio, as it was
observed for testing full-scale reinforced concrete beams in flexure (Babaeidarabad et al. 2014; Ombres 2011; Loreto et al. 2014). The average tensile loads of bonded four plies of FRCM composite to concrete blocks were only 1.25 times that tensile strength of bonded one ply of FRCM composite to concrete blocks due to the premature failure of FRCM composite. The comparison between the two test methods revealed that the ultimate tensile strength of the tested specimens under bending were much greater than that produced due to the pull-off test. In the pull-off tests, a pulling-off load was applied perpendicularly to the PBO fabric alignment, while in the bending test a direct tensile load was applied parallel to the PBO fabric alignment. That concluded the observed higher strength of FRCM composite in bending test rather than pull-off test.

CONCLUSIONS

An experimental study on the bond performance between externally applied FRCM composite and the concrete block is presented. The differences of the pull-off test results for various exposure conditionings were discharged due to partially scattering of the experimental data. The exposure conditionings did not influence the failure modes of FRCM composite. Variations in the applied load and the global slip were determined in the bending test, and the translation of the ultimate loads during delamination was experimentally observed. Failure mode of FRCM composite was observed to be highly dependent on the number of FRCM composite’s plies. The heavily and lightly concrete surface preparations coincided with respect to their ultimate tensile strength. The impermeable property of the cementitious matrix concluded that the saltwater or alkaline solution had no effect on the bond performance of FRCM composite. Higher pull-off strengths were determined for the specimens aged to 166 days in comparison with 72 days as the specimens continuously cured.
ACKNOWLEDGMENTS

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ABSTRACT

The innovative composite material Textile-Reinforced Concrete (TRC) opens up new application areas in structural concrete engineering and has many advantages for the building industry. By using non-corrosive technical textiles made of alkali-resistant glass (AR-glass) or carbon, the concrete covers can be minimized resulting extremely slender concrete members. In addition, they provide ecological and economic benefits, which can be recognized in the realized application projects. This paper summarizes selected filigree structures, which only could be fulfilled with textile-reinforced concrete. In this connection the self-supporting and bearing structures will be presented.

KEYWORDS

Filigree constructions, facades, pedestrian bridge, AR-Glass, Carbon, Textile-Reinforcement.

INTRODUCTION

During the last 30 years reinforced concrete surfaces were plastered increasingly or clad with aluminium or wood. This is due to its corrosion-prone reinforcement, which can lead to undesirable surface damage by concrete spalling. Consequently a corrosion resistant material such as textile-reinforced concrete inspires the renaissance of concrete surfaces. The use of the new composite material allows the production of thin-walled components, which are characterized by low weight and great slenderness. TRC is particularly suitable for self-supporting building structures such as facade plates (Figure 1). The conventional 90 mm thick reinforced concrete plates can be replaced by only 20 to 30 mm thin textile-reinforced concrete plates (e.g. Kulas et al. 2011). Another application is the use of textile-reinforced concrete for supporting structures with chloride attack. Existing steel reinforced bridges require high concrete cover to be able to protect the steel from carbonation and chloride attack. By application of high-strength and corrosion-resistant fabrics, the bulky structure can be counteracted. This leads to a revival of the concrete construction for shells (Figure 1).

SELF-SUPPORTING COMPONENTS

Small-sized facade

Smaller facade panels of TRC with element sizes up to about 2 m² and a thickness of about 20 mm are often used for ventilated facades. The low weight, the associated cost savings for transport and attachment and not least the
architectural design options are reflected in numerous applications in recent years. The applicability is especially promoted by the first officially approved textile concrete slab betoShell®.

**Large-sized facade**

Small-sized facade panels require a large number of fasteners per square meter facade surface, which represents a significant cost share of the total construction. Besides, such facades show a high joint portion, which is not always desirable from architectural view. Therefore, in collaboration with industry partners Hering Bau and solidian, large formatted tiles were developed at the Institute of Structural Concrete and the Institute of Building Materials of the RWTH Aachen.

The large-format facade panel with a panel thickness of 50 mm, which is about 6.2 square meters and was completed in 2014, is an architecturally sophisticated panel for the new business building "SchieferErlebnis" in Dormettingen (Figure 2). The size and the light weight reduce assembly time and the costs.

Constructively it concerns ventilated facade systems. The facade elements are fastened on the supporting wall and a thermal insulation is interposed between the two layers. Therefore the total thickness can be significantly reduced compared to conventional wall structures. The material-saving design with simultaneous gain in space inside the building is a good example of energy-efficient construction with reduced CO₂ emissions.

The facade panels have maximum dimensions of 1.5 m x 4.1 m with a panel thickness of only 50 mm. An efficient carboxylic textile, which achieved tensile stresses of about 3000 N/mm² in the composite body, was sufficient to ensure the load-bearing capacity (e.g. Hegger et al. 2007). To the anchorage fasteners, known from reinforced concrete, were modified for the small panel thickness.

In experimental studies, the load-bearing capacity of the facade panels was proved. To meet the demands of the client, like crack-free panels, the plate was calculated such that the tensile strength is not exceeded even at maximum wind load and temperature conditions. These high demands on the surfaces are important not only to satisfaction of the optics, but also for the conviction of the bearing capacity of the textile-reinforced concrete panels. Low deformations and crack-free construction parts in use give a feeling of safer constructions. Only in the ultimate limit state of bearing capacity (safety against failure), the textile reinforcement to ensure sustainability is activated. The good alliance between textile reinforcement and concrete and the high strength provide small crack openings and significant load increases after cracking until the break starts. With the help of the experiments it was shown that facade panels at failure after cracking can accommodate about three to four times the load on the used state.

![Figure 2 View on the facade of “SchieferErlebnis” in Dormettingen (Foto: solidian) and the used textile-grid, produced by the company solidian](image)

**SANDWICH FACADES**

Sandwich elements for industrial and multi-storey buildings are constructed primarily with layers out of metal or reinforced concrete now. Textile reinforced inner and outer shells compared to those made out of reinforced concrete reduce the cost of transport and fixing and composite medium. The lightweight sandwich structures can be used either for self-supporting, vertical Sandwich facades of multi-storey buildings and factories or as structural components for modular buildings.
Parts of the facade of a new Institute at RWTH Aachen were performed as self-supporting sandwich construction with dimensions of $L \times H \times T = 3.45 \times 1.00 \times 0.18$ m$^3$ (pitches) in an EU-promoted project (Figure 4). Basis of the planning were those in various literature presented results (e.g. Hegger et al. 2007; Shams et al. 2015; Shams and Stark 2013; Shams et al. 2014). The elements consist of a load-bearing PUR rigid foam core, two thin concrete shells, which are reinforced with glass fibre and composite needles made out of stainless steel. The weight is transmitted through the inner shell with welded on the post and beam construction consoles, the wind loads are taken up by four horizontal brackets (Figure 3). The weight of the outer shell is hung in the inner shell with diagonal anchors. To ensure permanent composite, special composite needles are used, which absorb the peeling normal force stresses in the bonding fugue that result from suction, temperature and shrinkage. The sandwich structural behavior and the stress of the outer layers are substantially determined by the rigidity of the insulating core according to the theory of elastic composite (e.g. Shams et al. 2014).

In four-point bending tests on prototypes a favourable load-deformation behavior was observed, which is limited by the shear capacity of the insulating core and which occupies the potential for greater spans. The cracking of the concrete layers can be suppressed over the use load range by textile reinforcement in conjunction with short fibres effectively. The positive experiences in planning, manufacturing and assembly demonstrate the high potential for application of textile reinforced concrete for use in facades.

Figure 3 Vertical and horizontal support of the edge elements including sealing strip

Figure 4 Longitudinal view on the south facade of the new building of the Institute INNOTEX
RECONSTRUCTION OF FILIGREE CONCRETE STRUCTURES

The innovative composite of Textile Reinforced Concrete offers new possibilities for reconstruction of historical buildings. Especially delicate components, such as the historic glass concrete windows of Aachen Town Hall, benefit from the corrosion-resistant and durable textile carbon reinforcement. The old windows had to be renovate because of corroded steel reinforcement and concrete spalling. To this the landmarked glass elements, which were embedded in reinforced concrete, were separated and installed again in the renovated windows. At the same time the steel reinforcement was replaced with carbon reinforcement. Consequently the monument editions were observed and the corrosion damages in future could be avoided.

As a part of preparations for the celebrations of Charlemagne in 2014, remediation measures were carried out at the Gothic town hall, which is part of the most important medieval stately homes of the German-speaking countries. Particularly the glass concrete windows, that should reflect on the idea of the architect Graubner the appearance of quarried stone masonry, are conspicuous (Figure 6). The windows vary in their appearance and size and consist of glass elements embedded in 3,0 cm thick concrete. The 2,5 cm wide concrete webs between the lines of glass elements were used for the steel reinforcement in the horizontal direction (Rempel et al. 2014). The windows are encased in steel frames that are required in the wall for the anchoring on the one hand and served in the production as a lost formwork to the other. Before preparing the new window, which took place on the same principle as already 60 years ago, a detailed documentation of the windows was held to restore the appearance in the reconstruction. So the original position of about 15.000 glass elements could be ensured. For this the separated elements of glass were positioned on a form panel inside the frame and then filled with concrete. Carbon rovings serve as reinforcement elements, which replace the old reinforcement bars (Figure 5). The possibility of realizing sharp components enables to dispense with the surrounding frame. So the 2 m long windows were divided into portable segments and could be assembled afterwards one above the other standing. This has both advantages in manufacture as well as during assembly. The sharp edges ensure that the stacked segments seem like a monolithic created window (Figure 6).

The bearing capacity tests show that the glass elements governing the use condition. In realistic tests, the glass failed due to compressive stresses before a crack in the concrete occurred. After the failure of the glass a ductile component behaviour ceased. The load capacity of the window was almost doubled to a breaking moment of 2,5 kNm/m, corresponding to a wind load of about 14 kN/m². By means of the combination of optically high-quality concrete of the company Hering Bau with the carbon textile of the company solidian, the desired permanent solution, that even could impress the critical preservationists, originated (Figure 7).
PEDESTRIAN BRIDGE

The world's first bridge made entirely from carbon-reinforced concrete was established in Albstadt-Ebingen on 22 October 2015 (Figure 8). The bridge was built without any steel-reinforcement or prestressed steel. Especially the absence of prestressed steel is what basically distinguishes the bridge from previous bridges made from textile reinforced concrete that can be found in Oschatz, Kempten and Albstadt-Lautlingen (Michler, H. 2013 and Hegger et al. 2011).

The complete absence of steel allows a bridge construction that is both low-maintenance and durable, as corrosion damage can be ruled out. Therefore, this solution is a promising and economic way of construction for communities and townships.

The bridge has a so-called trough cross-section, produced monolithically as a prefab. The centerpiece of the construction is the carbon reinforcement "solidian GRID", characterized by a breaking stress of over 3000 N/mm². As carbon reinforcements cannot corrode, very thin concrete covers can be used. The bridge was executed with a concrete cover of only 20 millimeters. This allows the construction to realize very slim components. The trough wall is only 70 millimeters thick, the pavement only 90 millimeters. Compared to a corresponding steel-reinforced construction, this translates into a more than 50 % weight reduction. With a length of 15 meters, the bridge weighs only 14 tons. It was designed for cyclists and pedestrians but can also accommodate a 5-ton snow-clearing vehicle.

On 23 February 2016 the company in charge received the innovation prize for structural concrete products of the supplier industry for the carbon-reinforced concrete pedestrian and cycle bridge built in 2015 (Figure 7). The award honors innovative products and services from the supplier industry to the precast concrete sector.

Alongside the solidian GmbH, responsible for the development and project management, also the municipality of Albstadt as builder-owner, the Max Bögl Bauservice GmbH & Co. KG as producer of the bridge, the Institute of Structural Concrete of the RWTH Aachen University as expert for the supporting construction, KnippersHelbig for the structural engineering calculation as well as the regional authority of Tübingen for the approval in individual cases were involved in this pilot project in Albstadt-Ebingen.

Figure 8 View of the TRC pedestrian bridge in Albstadt-Ebingen (Foto: solidian)
CONCLUSIONS

Structures made of textile-reinforced concrete can be carried out more clearly slimmer than steel components due to the corrosion-resistant technical textile. In addition to high load capacity, the TRC is characterized by a very finely divided crack image with very small crack widths, when the concrete tensile strength exceeded. This new composite material is particularly suitable for high demands on the concrete surface. This is impressively illustrated by the previously realized buildings. It is not only self-supporting elements such as facade panels that have been realized, but also viable filigree structures such as shells and bridges, which were successfully built.

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FRP Composites in New Construction
SHEAR TESTING OF GFRP BARS

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ABSTRACT

The high cost of reinforced concrete (RC) structures rehabilitation results in the increased utilization of non-metallic reinforcement. Glass Fibre Reinforced Polymer (GFRP) bars, due to the combination of relatively low cost (when compared to other composite materials) and a good durability, have become more popular than competitors (carbon or basalt polymer composite bars). Despite many advantages, the utilization of GFRP bars in reinforced concrete structures is still limited. This is due to the variability of GFRP bar properties among different manufacturers. To ensure wider utilization of this composite reinforcement, a better understanding of bars’ properties is needed. This research program is focused on GFRP bars’ shear properties, for both new and aged specimens. Three different types of bars from two different manufacturers were tested in shear. The aging of specimens was performed in a highly alkaline solution (pH of 13) heated to 60°C. In this aggressive environment bars were kept for 5 months. All test data were analysed and results are presented herein. The shear stress-strain relationships indicate a correlation between bars’ material properties (quality of resin, resin fiber interface, and fiber content) and the shear moduli.

KEYWORDS

Glass fiber reinforced polymers, GFRP, reinforced concrete, shear test, shear modulus, quality control.

INTRODUCTION

The usage of composite materials in civil engineering, especially FRP reinforcement, has grown significantly for the last two decades (Bakis 2002). However, when compared with conventional steel utilization of Glass Fiber Reinforced Polymer (GFRP) bars, it is still relatively limited. A reason for this can be found in the variability of mechanical properties and bars’ quality among different manufacturers. Moreover, due to patent protections and production confidentially details about resin properties or resin content are usually not available. Therefore, there is a need to establish practical and preferably simple testing protocols for the assessment of bar quality.

Shear testing of GFRP bars is relatively easy and has the potential to provide extensive information not only on bar shear capacity but also on material properties such as quality of resin, resin-fiber interface, or fiber content. During the test development additional information as, for example, shear modulus was omitted due to the nonlinearity of stress-strain relationship for direct shear (Gentry 2011). However, the analysis of the test results (shear curvatures) indicates that there is a characteristic repeatable pattern in each test outcome, which can be linked to different material properties. This research program is aimed at investigating this possibility to obtain additional information from the transverse shear test.

EXPERIMENTAL STUDY

Material description

Specimens tested in this study were provided by two companies (referred herein as Company I and Company II) operating in the Canadian market. All of the specimens were straight bars with similar diameters: #4 bars from Company I and 12M bars from Company II. Three different types of bars were tested: straight bars (SB) manufactured by the pultrusion process with additional surface treatment (Company I – sand coating; Company II – indented ribs), smooth surface bars (SSB) produced in a pultrusion process without additional surface treatment, and bent bars (BB) produced by a customized process individual for each company. Relevant material characteristics of all bar specimens as specified by the supplier and geometrical characteristics measured by us are
shown in Table 1. The nominal cross-sectional area and bar diameter are based on information provided by the supplier, while the effective cross sectional area and bar diameter were measured in this work.

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<td>mm</td>
</tr>
<tr>
<td>Effective bar diameter</td>
<td>14, 14, 12.7</td>
<td>12, 13.5, 12</td>
<td>mm</td>
</tr>
<tr>
<td>Effective bar diameter for shear</td>
<td>16, 14, 12.7</td>
<td>13.5, 13.5, 16</td>
<td>mm</td>
</tr>
<tr>
<td>strain computation</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nominal cross-sectional area</td>
<td>126.7, 126.7, 126.7</td>
<td>113, 143.06, 113</td>
<td>mm²</td>
</tr>
<tr>
<td>Effective cross-sectional area</td>
<td>153.85, 153.85, 126.7</td>
<td>113.05, 143.07, 113.06</td>
<td>mm²</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>65.6, 65.6, 41.9</td>
<td>60, 60, &gt;50</td>
<td>GPa</td>
</tr>
<tr>
<td>Transverse shear strength</td>
<td>212, 212, -</td>
<td>150, 150, -</td>
<td>MPa</td>
</tr>
<tr>
<td>Glass fiber content (by weight)</td>
<td>83, 83, -</td>
<td>&gt;85, &gt;85, -</td>
<td>%</td>
</tr>
</tbody>
</table>

**Testing procedures**

**Shear test**

Shear tests were performed according to Annex L of CSA S806-12 to determine transverse shear strength of FRP bars. Three different types of bars from both manufacturers were used in these tests: smooth surface bars (SSB) as control specimens, straight bars (SB), and straight portions of bent bars (BB). Specimens were divided into 6 sets according to bar type and diameter, and each set consisted of 8 samples. All specimens were 300 mm long. Double shear was applied to each specimen using a special shear test device (Figure 1), with stress applied at 45 MPa/min. In order to achieve this uniform stress rate and due to different bar diameters, the load control rate varied for each bar size as shown in Table 2.

<table>
<thead>
<tr>
<th>Company I</th>
<th>Company II</th>
</tr>
</thead>
<tbody>
<tr>
<td>SSB and SB</td>
<td>#4 Load rate 14 kN/min</td>
</tr>
<tr>
<td>BB</td>
<td>#4 Load rate 10 kN/min</td>
</tr>
</tbody>
</table>

The shear test device consists of two bar holders, two bottom blades unique to the outside diameter of each bar, and guides that were used to create distance between the bottom blades. All parts are connected together by threaded rods. Shear load is applied to the bar by the upper blade connected to the machine load cell.

![Figure 1 Shear test device with a sample a) before testing, b) after testing](image)

**Accelerated aging test**

The alkaline immersion test was chosen for accelerated aging. Testing methodology used in this research was developed according to the CSA S806 – 12 standard Annex M, and supplemented with information included in ASTM D7205/D7205M protocol. Bars were kept in a highly alkaline solution (pH 13, 118.5g of Ca(OH)₂, 0.9g of NaOH and 4.2g of KOH in 1 L of deionized water) heated to 60°C for five (5) months. After this period of time, bars were taken out from the bath and tested in shear.
RESULTS AND DISCUSSION

The primary test result is the nominal transverse shear stress, calculated as half of the peak failure load divided by the cross-section area (effective cross-section area reported in Table 1). The shear test consists of cutting the bar through two parallel cutting planes. Different failure modes can be observed in these tests; when both planes fail at the same time and when one plane fails after the other. The typical stress-vertical displacement curves for these cases are shown in Fig 2a and 2b.

![Failure of both planes](image1)
![Failure of the first plane](image2)

**Figure 2** Shear failure modes a) double shearing at the same time b) planes shearing consecutively

Stress-strain relationships were determined and the comparison of the most typical curves for each type of bar is shown in Figure 3a for Company I, and Figure 3b for Company II, respectively. The stress-strain relationships are calculated as stress (load divided by effective cross sectional area) versus strain (displacement divided by the effective diameter). From the obtained shear stress-strain curves (Figure 3) for all tested specimens it can be seen that the shear stiffness (shear modulus) is stress dependent. Some statistical analysis indicators are shown in Table 3 for strength and Table 4 for shear modulus.

![Stress-strain curves for bars from Company I](image3)
![Stress-strain curves for bars from Company II](image4)

**Figure 3** Stress-strain curves a) Company I b) Company II

The differences in the stress-strain relationships for bars produced by pultrusion (straight bars SB and SSB) and bars produced by customized methodology (bent bars BB) indicate that the shapes of these curve can reflect materials’ characteristics. Bent bars experience higher deformations than straight bars. This can be due to the application of different resins in bent bars’ production process as well as lower fibre content in these bars. This is true for both companies, which use different, then both straight bars, resins and lower fibre content in bent (BB) bars.

Further investigation of stress-strain relationships in shear indicates that all curves share the same characteristics: regions with similar stress-strain behaviour. Each curve can be divided into three different sections: two sections with a relatively stable shear modulus (G1 and G2) and a section with a characteristic “plateau” between two moduli, were strains increase more rapidly (Figure 4).
Table 3 Shear test results – Strength [MPa]

<table>
<thead>
<tr>
<th>Properties</th>
<th>Company I</th>
<th>Company II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Size</td>
<td>#4</td>
<td>M12</td>
</tr>
<tr>
<td>Bar type</td>
<td>SB</td>
<td>SS</td>
</tr>
<tr>
<td>Mean [MPa]</td>
<td>242</td>
<td>195</td>
</tr>
<tr>
<td>S.D.</td>
<td>24</td>
<td>1.7</td>
</tr>
<tr>
<td>C.o.V</td>
<td>0.1</td>
<td>0.01</td>
</tr>
</tbody>
</table>

The shear secant moduli were calculated for the two regions: the first before the plastic plateau, and the second after the plastic plateau (Figure 4). The results are reported in Table 4. The standard deviation (S.D.) and coefficient of variation (C.o.V) are from testing 8 samples for each case reported in Table 4.

Table 4 Shear test results – Shear modulus [GPa]

<table>
<thead>
<tr>
<th>Properties</th>
<th>Company I</th>
<th>Company II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Size</td>
<td>#4</td>
<td>M12</td>
</tr>
<tr>
<td>Bar type</td>
<td>SB</td>
<td>SS</td>
</tr>
<tr>
<td>Modulus</td>
<td>G1</td>
<td>G2</td>
</tr>
<tr>
<td>Mean [GPa]</td>
<td>1.45</td>
<td>2.02</td>
</tr>
<tr>
<td>S.D.</td>
<td>0.04</td>
<td>0.13</td>
</tr>
<tr>
<td>C.o.V</td>
<td>0.03</td>
<td>0.07</td>
</tr>
</tbody>
</table>

One can see that not only a different type of bar (SB and SSB vs. BB) but also a different type of surface finishing (sand coating and indented ribs vs. smooth surface bars) have influence on shear moduli. The largest values of moduli were obtained for smooth surface bars (SSB) for both companies. Moreover, SSB bars are also characterized by the smallest difference between two moduli (G1 and G2) with slightly higher values for the second shear modulus. In the case of straight bars (SB), samples from Company II are characterized by the same behaviour as SSB samples, while SB bars from Company I show a smaller G1. This fact indicates that sand coating surfaces have a larger influence on a shear modulus than indented ribs. This makes sense since indented ribs do not introduce a new material to the sample (as sand coating does), but they simply change the sample’s shape. The smallest shear modulus was obtained for bent bars (BB) from both companies, where G2 in this case shows the larger value.

Based on the obtained values for shear moduli and the assumptions regarding the mechanics of shearing process, it is assumed that the first shear modulus (G1) indicates resin properties, while the second one (G2) can be linked to the properties of fibers, and the plastic plateau between both moduli occurs due to the delamination process between the resin and the fibers. To investigate this speculation further, an additional shear test was performed on chemically aged bars. The bars were immersed in an alkaline solution (pH 13) for 5 months at 60°C. Since the highly alkaline solution is more severe for the glass than for the polymeric resin, it was assumed that the modulus connected with the properties of fibers (G2) would degrade more than the modulus linked with the resin properties (G1). Some statistical indicators for the alkaline aged samples are shown in Table 5 for strength and Table 6 for shear modulus.
Table 5 Shear test results – Strength for aged samples [MPa]

<table>
<thead>
<tr>
<th>Properties</th>
<th>Company I</th>
<th>Company II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Size</td>
<td>#4</td>
<td>M12</td>
</tr>
<tr>
<td>Bar type</td>
<td>SB</td>
<td>SB</td>
</tr>
<tr>
<td>Mean [MPa]</td>
<td>215</td>
<td>205</td>
</tr>
<tr>
<td>S.D.</td>
<td>4.5</td>
<td>1</td>
</tr>
<tr>
<td>C.o.V</td>
<td>0.02</td>
<td>0.005</td>
</tr>
</tbody>
</table>

Table 6 Shear test results – Shear modulus for aged samples [GPa]

<table>
<thead>
<tr>
<th>Properties</th>
<th>Company I</th>
<th>Company II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Size</td>
<td>#4</td>
<td>M12</td>
</tr>
<tr>
<td>Bar type</td>
<td>SB</td>
<td>SB</td>
</tr>
<tr>
<td>Mean [GPa]</td>
<td>1.22</td>
<td>1.63</td>
</tr>
<tr>
<td>S.D.</td>
<td>0.01</td>
<td>0.02</td>
</tr>
<tr>
<td>C.o.V</td>
<td>0.01</td>
<td>0.04</td>
</tr>
</tbody>
</table>

It is noticeable that both moduli (G1 and G2) degrade significantly (see also Table 4). Comparisons of shear moduli before and after the alkaline immersion test are shown on Figure 5 a) for Company I and b) for Company II. The percentage decay of shear modulus is shown in Table 7.

Figure 5 Shear moduli comparison of samples before and after 5 months of aging a) Company I b) Company II

Table 7 Percentage decrease in shear modulus for samples after accelerated aging test [%]

<table>
<thead>
<tr>
<th>Properties</th>
<th>Company I</th>
<th>Company II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Size</td>
<td>#4</td>
<td>M12</td>
</tr>
<tr>
<td>Bar type</td>
<td>SB</td>
<td>SB</td>
</tr>
<tr>
<td>Change in Moduli</td>
<td>15.7 32</td>
<td>36.1 40.2</td>
</tr>
</tbody>
</table>

Table 7 shows that, as expected, the shear moduli G2 deteriorated more than shear moduli G1. The most deterioration happened for company I for bars with smooth surface finishing. These bars do no have the beneficial protection of sand coating and thus are more susceptible to the aggressive environment. In the case of bars from Company II, both smooth bars (SSB) and straight bars (SB) do not have any additional coating and thus their deterioration in alkaline environment is similar. The smallest deterioration process occurred in the case of bent bars (BB) from Company II, which can be due to the additional polyethylene sleeve placed on bars during the production process, which works as an additional protection (similar to the sand coating on SB bars from Company II) against the alkaline solution.

As expected, all samples experienced the highest stiffness decrease for the second shear modulus (G2) (Table 7); the modulus that is influenced most by the glass fibers properties. Glass fibers are more susceptible to deterioration in the alkaline environment then the polymeric resin. This will effectively decrease the values of the G2 moduli.
CONCLUSIONS

The research program presented in this paper is focused on shear testing of GFRP bars and information that can be obtained from such testing. The results of the shear testing are reported in terms of shear strength and stiffness moduli. The shear stiffness varies, depending on the level of stress, but shear stress-strain relationships for all tested bars show certain similarities. All stress-strain relationships obtained in this research can be divided into three distinguishable regions: one linked with resin properties G1, one with fibre properties G2, and the third region, a plateau between the moduli G1 and G2, which is likely due to the delamination process.

In the comparison of the two bar types, bars (from both companies) produced by pultrusion (straight bars SB and SSB) versus bars from the customised production process (bent bars BB), show the shear test can be potentially used as an indicator for resin properties and fibre content/properties. The analysis of the results from accelerating aging tests confirm the speculation made regarding the shear stress-strain responses of the bars and their link with matrix and fibre properties.

The results presented in this paper indicate the shear test has a significant potential in determining FRP bars properties. This potential is not directly indicated in the current testing protocols (Annex L of CSA S806-12). Further research is needed for better understanding of the phenomenon and better utilization of the described shear testing for quality control of the GFRP bars.

REFERENCES


CSA S806-12 “Design and construction of building structures with fiber – reinforced polymers”

CSA S807-10 “Specification for fiber – reinforced polymers”

EXPERIMENTAL INVESTIGATION OF THE SHEAR RESISTANCE OF GFRP COMPOSITE BARS

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ABSTRACT

Glass fiber reinforced polymer (GFRP) composite bars are considered durable alternative to regular steel bars for reinforcing concrete members, in corrosive environments. In concrete pavements, dowel bars are used to transfer loads between adjacent panels across construction, control, and expansion joints. Such load transfer occurs through shear action in the dowel bar. Steel dowel bars tend to corrode prematurely. GFRP bars have high corrosion resistance, however, the transverse shear properties of GFRP bar vary widely, depending on their chemical and physical properties. This paper presents comprehensive review of literature along with experimental investigation of GFRP bars for use as dowel bars in concrete pavement, at the construction and control joints. Two series of GFRP bars having diameters of 16mm and 32 mm were investigated and reported.

KEYWORDS

GFRP bars, concrete pavement, control joints, construction joints, shear resistance.

INTRODUCTION

Corrosion of internal reinforcing steel is one of the most important factors that lead to premature deterioration of concrete structure. The steel rebar in the concrete structure is generally protected by the alkalinity of the concrete to prevent corrosion. However, the area with aggressive environment, such as suffering heavy precipitation and snow, high moisture content and large amount of chlorides ions in deicing salts could reduce the alkalinity of the concrete and result in the corrosion of reinforcing steel bars and cracks in concrete. Corrosion can increase the cost of maintenance and reduce the service life of the structure. Glass Fiber Reinforced Polymer (GFRP), as a non-corroded material, is currently prompted for the industry.

In concrete pavements, dowel bars are used to transfer loads between adjacent panels across construction, control, and expansion joints. Such load transfer is secured through shear action in the dowel bar. Steel dowel bars tend to corrode prematurely, and cause deterioration of the concrete pavement joints. In order to retrofit concrete pavement with corroded steel dowel bars, GFRP composite bars are considered durable alternative for replacement of corroded steel dowel bars. GFRP bars have high corrosion resistance, however, the transverse shear properties of GFRP bar vary widely, depending on their chemical and physical properties. Investigation for shear properties of GFRP bars with different diameters will be conducted by literature reviews and GFRP bar shear resistance tests.

DOWEL ACTION

When two concrete slabs move relative to each other, the dowels are subjected to a shearing action that can be defined as dowel action. Dowel action is the secondary mechanism to resist the applied shear force when dowel bars are across a joint.

Figure 1: Deflected shape of dowel bar (Vijay 2009)
Load transfer efficiency (LTE) and Joint efficiency (E) are normally used to evaluate the performance of the dowel bar by deflections. The equation for LTE and E are shown as below,

\[
LTE = \frac{d_u}{d_l} \times 100\%
\]

\[
E = \frac{2d_u}{d_l + d_u} \times 100\%
\]

where,

\(d_u\) = deflection of the unloaded side of a joint; and

\(d_l\) = deflection of the loaded side of a joint.

According to AASHTO standards, the value of LTE is between 70 and 100 represents very good load transfer. (Vijay 2009) ACPA recommends that the value of E should be equal or greater than 75% in pavement joints. (Benmokrane et al. 2014)

**LITERATURE REVIEWS**

**Performance of GFRP Dowels for Concrete Pavements**

Eddie conducted a study on the performances of GFRP dowel bars for concrete pavements. Eddie tested three groups of GFRP bars manufactured by three different U.S. suppliers and one group of epoxy steel bars. Each group contains 3 samples. Double shear test was applied on the first two groups of GFRP dowels and the group of epoxy steel in the laboratory, and Falling Weight Deflectometer (FWD) Tests were applied in the field to record the deflections for all three groups GFRP dowels and one group steel dowels. FWD Tests were applied in the field 8 months after it was opened for traffic. The diameter of GFRP dowels is 38mm, and the diameter of epoxy steel is 31.75mm. According to the test results, shear strength of GFRP dowel is much lower than steel dowel. However, two groups of GFRP dowel bars performed similar or better than steel dowels result in joint efficiency analysis. The third group of GFRP dowel did not perform as well as the previous two groups. However, the performances were still qualified. The second group of GFRP dowels (GFRP type-2) with larger shear strength than the first group of GFRP dowels (GFRP type-1) performed better. The only concern is that GFRP dowels suffer larger deflections than steel dowels under dynamic loading conditions, which means GFRP dowels have lower joint stiffness. Also, GFRP dowels require larger diameters compared with steel dowels to achieve the same or better performance. Therefore, larger diameters and lower stiffness can reduce the bearing stress between concrete and GFRP dowels. Since bearing stress is one of the major failure causes at joints, GFRP dowels could be a better choice. Furthermore, there are no strict limitations for GFRP dowels at construction design. The larger deflection for GFRP dowels is no more a concern for applying at joints. (Eddie et al. 2001) Murison indicated in the research “Evaluation of GFRP Dowels” that Eddie performed FWD tests in the field after two years of traffic, and there is no big change on results. (Murison 2004)

![Figure 2 Joint efficiency for steel dowels and GFRP dowels in FWD test (Eddie et al. 2001)](image)

<table>
<thead>
<tr>
<th>Dowel type</th>
<th>Dowel diameter (mm)</th>
<th>No. of samples</th>
<th>Ultimate double shear load (kN)</th>
<th>Mean shear strength (MPa)</th>
<th>Standard deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Epoxy-coated steel</td>
<td>31.75</td>
<td>3</td>
<td>901</td>
<td>570.00</td>
<td>8.8</td>
</tr>
<tr>
<td>GFRP type-1</td>
<td>38.10</td>
<td>3</td>
<td>244</td>
<td>107.00</td>
<td>3.8</td>
</tr>
<tr>
<td>GFRP type-2</td>
<td>38.10</td>
<td>3</td>
<td>343</td>
<td>150.00</td>
<td>3.2</td>
</tr>
<tr>
<td>GFRP type-3</td>
<td>38.10</td>
<td>3</td>
<td>231</td>
<td>101.30</td>
<td>3.1</td>
</tr>
</tbody>
</table>

Table 1 Shear strength tests for steel dowels and GFRP dowels (Eddie et al. 2001)
GFRP BAR SHEAR TESTS

Objective & Standard

The objective of this test is to determine the ultimate transverse shear strength of φ16mm and φ32mm ComBAR GFRP rebar. The test was conducted according to the procedures recommended by the ACI 440.3R-04 Committee report, section B.4 entitled “Test method for transverse shear strength of FRP bars” (ACI 440.3R-04).

For testing φ16mm ComBAR rebar, the shear test device was according to ACI 440 report, however, it had to be modified to accommodate testing φ32mm rebar. The current ACI test method presents recommended dimensions for the shear test device for testing FRP rebar of diameter up to #8 (25mm). To ensure complete shearing of the φ32mm ComBAR, the top ½ inch of the upper blade was cut before testing. The base and the blades of the shear device were made of mild (non-hardened) steel.

Equipment & Test Load Rate

A Universal compression-testing machine was used to conduct the shear test. The capacity of the testing machine is 300 kips (1334.5 kN). For the φ16mm ComBAR, the load was increased at a rate of 5.0 kips/min. (22.24 kN/min.), which is equivalent to an increase in shear stresses at a rate of 8.0 ksi/min (55.3 MPa/min). For the φ32mm ComBAR, the load was increased at a rate of 20.0 kips/min. (88.96 kN/min.), which is equivalent to an increase in shear stresses at a rate of 8.0 ksi/min. (55.3 MPa/min).

Test Procedure

Figures 3 through 11 show the ComBAR test specimens before, during and after testing. A total of seven φ16mm and seven φ32mm ComBAR specimens were tested. Figure 3 shows the test specimens.

While the ACI recommended shear test device accommodates well a φ16mm ComBAR, it needs slight modifications to accommodate the φ32mm ComBAR, as shown in Figures 5. Notice that the top of the rebar is higher than the top of the base of the shear device. Additional spacers were provided to rise the attach plates to allow for attaching the φ32mm ComBAR to the shear test device, as shown in Figure 6.

Figure 7 and 8 show a φ16mm ComBAR specimen during the test and after failure. For all the φ16mm ComBAR rebar specimens, the mode of failure was in the form of complete shearing of the rebar, as shown in Figure 8.

Figure 9 shows the φ32mm ComBAR during a test and after failure. Load was applied until complete shearing off of the ComBAR test specimens. For all the φ32mm ComBAR rebar specimens, the mode of failure was in the form of shear failure associated with partial disintegration of the 1.0 inch (25mm) wide sheared section of the test
specimen. Figure 10 shows a close-up of a sheared center section for a φ32mm ComBAR specimen. In Figure 11, a φ 32mm ComBAR rebar test specimen at failure.

Figure 5 φ32mm ComBAR on the shear device

Figure 6 φ32mm ComBAR inside the shear device
(Notice the additional spacers to raise the attach plates)

Figure 7 φ16mm ComBAR rebar inside the testing machine

Figure 8 A φ16mm ComBAR rebar after failure

(a) A φ32mm ComBAR rebar during testing

(b) A φ32mm ComBAR rebar at failure

Figure 9 A φ32mm ComBAR rebar test specimen during testing and after failure

Figure 10 A sheared section of a φ32mm ComBAR rebar test specimen

Figure 11 A φ32mm ComBAR rebar test specimen at failure

All specimens were tested at room temperature, at 77 °F (25 °C), and relative humidity of 47%.

Test Results

Data Analysis

During the test, the applied load was increased at a rate such that the shear stresses increased at a rate of 8.0 ksi/min (55.3 MPa/min). The ultimate applied load was recorded. Table 1 shows the test results for both the φ16mm and φ32 mm ComBAR specimens. The shear stress was determined using the following equation (3):
\[
\tau_u = \frac{P_s}{2A}
\]  

(3)

where
\[\tau_u = \text{shear strength, ksi (also given in MPa in Table 1);}\]
\[P_s = \text{maximum failure load, kips;}\]
\[A = \text{cross-sectional area of specimen, in}^2.\]

The cross-sectional area was based on the reported diameter of the specimens; 16mm, and 32mm for \(\phi_{16}\)mm and \(\phi_{32}\)mm ComBAR rebars, respectively.

<table>
<thead>
<tr>
<th>Specimen #</th>
<th>Load Shear stress</th>
<th>Shear stress</th>
<th>Load</th>
<th>Shear stress</th>
<th>Shear stress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(lbs.) (ksi)</td>
<td>(MPa)</td>
<td>(lbs.) (ksi)</td>
<td>(MPa)</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>18750 30.08</td>
<td>207.42</td>
<td>58500 23.46</td>
<td>161.78</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>21750 34.90</td>
<td>240.60</td>
<td>57100 22.90</td>
<td>157.91</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>18500 29.68</td>
<td>204.65</td>
<td>60000 24.07</td>
<td>165.93</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>22500 36.10</td>
<td>248.90</td>
<td>62250 24.97</td>
<td>172.16</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>18900 30.32</td>
<td>209.08</td>
<td>69600 27.92</td>
<td>192.48</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>23250 37.30</td>
<td>257.20</td>
<td>63500 25.47</td>
<td>175.61</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>19000 30.48</td>
<td>210.18</td>
<td>67300 26.99</td>
<td>186.12</td>
<td></td>
</tr>
</tbody>
</table>

Table 2 Load vs. shear stress for \(\phi_{16}\)mm and \(\phi_{32}\)mm ComBAR rebar

Based on the test results, the average shear strength of the \(\phi_{16}\)mm and \(\phi_{32}\)mm ComBAR specimens were 225.43 MPa, and 173.14 MPa, respectively. Considering a guaranteed strength as
\[V'_{u} = V_{avg} - 3(\text{Standard deviation})\]
would lead to guaranteed shear strength for \(\phi_{16}\)mm and \(\phi_{32}\)mm of 157.84 MPa and 135.13 MPa, respectively. However, given the fact that in GFRP reinforced concrete members the concrete completely surrounds the GFRP rebar, which would result in much higher GFRP shear resistance (dowel action). Therefore, the \(\phi_{16}\)mm and \(\phi_{32}\)mm ComBAR should easily exhibit shear resistance of at least 200 MPa and 150 MPa, respectively.

**Test Observations**

According to the opinion of the Principal Investigator, the shear test results of the GFRP rebars following the recommendations of the ACI 440.R3-04 report may be affected by the followings. The diameter of the semi-circles of the upper and lower shear blades may affect the mode of failure as well as the ultimate shear load. Figure 12 shows a standard upper and lower shear blades. As shown in Figure 12, the shear failure of GFRP rebar is associated with transverse (horizontal) tensile strains, and deformation of the rebar, which results in very high shear stresses near the center of the rebar and very low stress away from the center of the rebar. As GFRP rebar is transversely un-reinforced, it may experience physical disintegration during the test. Figure 13 shows upper and lower blades with semi-circles having diameters equal to that of the GFRP rebar. As the GFRP rebar cannot deform laterally, as shown in Figure 13, it should exhibit much higher shear resistance. It is important to note here that inside a reinforced concrete member, a GFRP rebar is surrounded by concrete, similar to the situation shown in Figure 13.

![Figure 12 Side-view of the upper and lower blades (with standard semi-circles)](image1)

![Figure 13 Side-view of the upper and lower blades (diameter of semi-circles equal to diameter of GFRP rebar)](image2)
Discussion

According to the data obtained in the study “Performance of GFRP Dowels for Concrete Pavements” and the shear test above, shear strength comparison for GFRP dowels with different diameters and epoxy-coated steel dowels can be summarized in the table shown below.

Table 3 Shear strength for steel and GFRP dowels

<table>
<thead>
<tr>
<th>Dowel type</th>
<th>Diameter (mm)</th>
<th>Shear strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Epoxy-coated steel</td>
<td>31.75</td>
<td>570</td>
</tr>
<tr>
<td>GFRP</td>
<td>16</td>
<td>225.43</td>
</tr>
<tr>
<td>GFRP</td>
<td>32</td>
<td>173.14</td>
</tr>
<tr>
<td>GFRP</td>
<td>38.1</td>
<td>119.43</td>
</tr>
</tbody>
</table>

Table 3 shows that epoxy-coated steel dowels have much higher shear strength than GFRP dowels. Even with similar diameter, the shear strength for steel dowel is more than three times higher than GFRP dowels. Furthermore, GFRP dowels with smaller diameters obtain higher shear strength. Through thickness properties variations, similar phenomenon has been also observed in tensile strength test. In addition, a modified non-standard shear test method for large diameter GFRP bars has been developed, as the standard test is not suitable for large diameter GFRP bars.

CONCLUSION

According to the other related research and the shear strength test obtained above, it can be concluded that GFRP dowel bar has much lower shear strength than steel dowel bars. However, the performance of GFRP dowels with larger diameter is similar or better than steel dowels under service loads according to the calculations for joint efficiency from the research conducted by Eddie. Therefore, GFRP dowel bar with larger diameter, as a non-corroded and low maintenance material, can be a good alternative for steel dowel bar in concrete pavement. Furthermore, non-standard shear test method for larger diameter bar has been developed.

ACKNOWLEDGMENT

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BEARING STRENGTH OF SMALL-DIAMETER SELF-TAPPING FASTENERS IN HYBRID STEEL-FRP CONNECTIONS

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School of Architecture¹
School of Civil Engineering²
Georgia Institute of Technology, USA

ABSTRACT

This paper reports on the bearing strengths of small-diameter self-tapping fasteners used to join pultruded FRP composite structural sections to thin-gage steel. Single- and double-lap shear specimens were tested with 13 fastener group patterns to assess the bearing strengths of the pultruded composite material. In addition, a set of experiments using smooth pins matching the minor diameter (4.0 mm) and major diameter (5.6 mm) of the self-tapping fasteners was completed to assess the impact of the screw threads on the composite material. These experiments indicate that the effective diameter of the self-tapping fastener can be taken as the shank diameter of the screw.

The research finds that bearing failure is the dominant failure mode in the zero degree direction for connections with up to four fasteners in a group. In 45 and 90 degree connections, a transition to net-tension failure occurs with only two fasteners in the group. The tests demonstrate that self-tapping fasteners are an economical means of joining composite materials to steel, especially for thin-wall sections of FRP and steel.

KEYWORDS

Pultruded FRP, mechanical fastening, self-tapping fasteners, bearing capacity.

INTRODUCTION

Structural members using pultruded FRP composites are produced in open and closed sections, with wall thicknesses ranging from 2 to 20 mm. Connections in larger pultruded members, with relatively thick webs and flanges, and cross-sections similar to those of hot-rolled steel members, are typically achieved with large-diameter steel bolts. The connecting elements between the FRP members are typically made of mild or stainless steel. The behavior of these bolted connections is reflected in the ASCE pultruded FRP design standard currently being developed in the United States (Mottram, 2013).

Smaller pultruded sections with thin walls are often used in applications similar to those of cold-formed steel members, such as load-bearing stud walls and roof trusses. In cold-formed steel, bolts are rarely used. Rather, most connections are completed with self-tapping fasteners (She et al., 2012). In this paper, the bearing strength of small-diameter self-tapping fasteners in FRP-steel connections is assessed, and the applicability of this fastening technique for use in FRP pultruded structures is discussed. The connection method is advantageous for these smaller-size members because the connections can be completed quickly using a torque-limiting driver/drill. Inherent in the method is the ability to work from one side of the connection, without the need to hold a nut on the back side of the connection (i.e., blind fastening). Properly design connections using this method tend to be self-aligning, as the fastening process removes a modest amount of the material from both the steel cleat and the FRP members. Applications for this fastening technique are discussed in prior work by the first author describing the design of a lightweight composite structures for photovoltaic racking system (Gentry et al., 2014).

The two preferred failure modes for mechanically-fastened FRP connections are (1) net section failure, which is generally difficult to achieve due to the large number of fasteners required, but maximizes the efficiency of the
connection and (2) bearing failure in the FRP material, which is generally provides a predictable and ductile failure mode (Bank, 2013, Bank and Arora, 2007). The shear-out, block shear and cleavage failure modes tend to occur suddenly, and are less easy predict. For this reason, edge distance and fastener spacing requirements, related to the fastener diameter and FRP plate thickness, have been established to preclude these failure modes (Tajeuna et al., 2016). Bolt rupture and pull-through failure modes are prevented by proper sizing of the bolts and washers (Mottram and Turvey, 2003).

Most of the research on the use of small-diameter fasteners in FRP composites focuses on the bonding of pre-cured plates to concrete or steel structures as a repair technique. Nevertheless, some of this work is relevant to the current study. Lamanna et al. (2004) developed a method to fasten FRP strips to concrete beams using powder-actuated fasteners and observed that the FRP strip was less likely to detach from the concrete if the FRP was predrilled before fastening.

DESIGN OF JOINTS IN PULTRUDED FRP MEMBERS USING SELF-TAPPING FASTENERS

The joint design for the FRP-steel connections developed in this research are based on the use of thin-wall steel web cleats or web splices to transfer forces across the member boundaries. Two examples of these connections are shown in Figure 1.

Figure 1 Pultruded FRP to steel connections in photovoltaic panel frame

Configuration and Installation of Self-Tapping Fasteners

The bearing condition of the self-tapping fastener used in this research differs from the conventional case of a bolt in pin-bearing for the following reason. First, the fastener installation method creates tension in the fastener and thus lateral restraint to the joint. The integral washer may add to this lateral restraint. It is likely that the joint will have higher initial stiffness than a connection in pure pin bearing, due to the torque in the fastener. As the connection is loaded it will slip into bearing as the load overcomes the friction caused by the bolt (Olmedo and Santiuste, 2012). Second, the threads of the fasteners are necessarily in the plane of the FRP material, which is generally not recommended in the case of bolted connections (Mottram and Zafari, 2011). Finally, there may be little “clearance hole effect”, because the self-tapping fastener creates a tight interface with the FRP material as it drills through the FRP. The magnitude of this effect is an important aspect of the research.

For the No. 12 fasteners used in this research, a 4 mm pilot hole in the steel and a 5 mm pilot hole in the FRP were drilled or waterjet cut. These holes were selected based on the major diameter, 5.5 mm, and minor diameter, 4.0 mm, of the No. 12 self-tapping screw. The self-tapping fastener was driven carefully with a manual torque-limiting driver to a torque of 9 N-m. The initial driving caused the threads to form in the steel. As the driving continued the parts were drawn together, and the torque in the fastener rose to the specified level. If the screw is over-driven,
two failures are possible: first, the mild steel will strip and the fastener will lose its pre-setting tension and two, the fastener will fail in torsion. For the fasteners used in this research, the failure torque is around 20 N-m. Figure 2 depicts the threading and driving torque for the fastening of a 3 mm thick FRP plate to a 1.6 mm hot-rolled mild steel plate.

![Figure 2 Self-tapping fastener torque versus slack take-up in FRP-steel joint](image)

**Observation on Fastener and Material Selection**

In addition to self-tapping fasteners, a set of preliminary experiments were conducted using self-drilling fasteners, which have an integral drill bit and do not require a pilot hole. It was found that the self-drilling fasteners tended to over-drill the FRP material, leaving a large void around the circumference of the fastener in the FRP which led to reduced bearing capacity. The 1.6 mm thick (16 gage U.S.) hot-rolled steel plate was found to be the thinnest plate which would consistently self-thread and hold the self-tapping screws. Thicker plates worked as well or better (in terms of thread forming and screw engagement) but were more than required to carry the in-plane shear loads for the connections. Though the bearing strength data below includes tests on single and two screw connections, it was found that a minimum of four screws were needed to fully-secure an FRP-steel joint using self-tapping fasteners.

**BEARING TESTING PROGRAM**

In order to assess the bearing capacity of the FRP-steel connections, a series of lap shear tests were completed. The following aspects of bearing behavior were considered important in the testing:

1. The direction of the bearing stress relative to the longitudinal axis of the composite (the direction of pultrusion).
   Tests were completed in the 0°, 45° and 90° directions.
2. The number of and pattern of fasteners used to join the FRP to the steel plate. Lap shear specimens with one to six fasteners were tested.
3. Pin-bearing resistance of smooth shank bolts of with the same diameters as the minor and major diameter of the self-tapping fasteners.
4. Effect of clamping force in the fasteners.
5. Retention of bearing capacity in hot-wet environments (not discussed in this paper).

In the discussion that follows, the bearing strength values presented here are provided as forces and not stresses, as it is not clear what diameter to use for normalizing the bearing force into a bearing stress. The bearing strength is taken as the peak force – and not the 2% bearing offset force from ASTM D5691 (2013) or the 4% offset from ASTM D953 (2010). The material properties and bearing test data represent an average of 5 specimens unless otherwise noted. Further discussion on offset bearing strength is provided with the test results, below.

**FRP Material Properties**
The pultruded FRP materials used in the test were nominal 3.2 mm thick, E-glass reinforced with layers of uni-directional roving and continuous filament mats and a fire-retardant thermosetting polyester resin. The mechanical properties are given in Table 1.

<table>
<thead>
<tr>
<th>Property</th>
<th>Notation</th>
<th>Mean</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal Modulus of Elasticity</td>
<td>$E_x$</td>
<td>16.0 GPa</td>
<td>7.5%</td>
</tr>
<tr>
<td>Longitudinal Tensile Strength</td>
<td>$\sigma_x$</td>
<td>264 MPa</td>
<td>3.2%</td>
</tr>
<tr>
<td>Open Hole Tensile Strength, Long.</td>
<td></td>
<td>224 MPa</td>
<td>7.4%</td>
</tr>
</tbody>
</table>

The laminate structure of the FRP was determined by a resin burnout test according to ASTM D2584 (2011) and is given in Table 2. The volume fractions of the veil, CFM (continuous filament mat) and unidirectional layers were estimated and fiber weights of each ply were used to determine the effective ply thickness.

<table>
<thead>
<tr>
<th>Constituent/Ply</th>
<th>Material</th>
<th>Volume Fraction</th>
<th>Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fiber</td>
<td>E-glass veil, mat and unis</td>
<td>30%</td>
<td>3.05 mm</td>
</tr>
<tr>
<td>Resin</td>
<td>Unsaturated polyester</td>
<td>67%</td>
<td></td>
</tr>
<tr>
<td>Filler</td>
<td>Unknown</td>
<td>3%</td>
<td></td>
</tr>
<tr>
<td>Ply 1</td>
<td>Glass Veil</td>
<td>15%</td>
<td>0.29</td>
</tr>
<tr>
<td>Ply 2</td>
<td>CFM</td>
<td>20%</td>
<td>0.55</td>
</tr>
<tr>
<td>Ply 3</td>
<td>Unidirectional Rovings</td>
<td>65%</td>
<td>0.35</td>
</tr>
<tr>
<td>Ply 4</td>
<td>CFM</td>
<td>20%</td>
<td>0.44</td>
</tr>
<tr>
<td>Ply 5</td>
<td>Unidirectional Rovings</td>
<td>65%</td>
<td>0.35</td>
</tr>
<tr>
<td>Ply 6</td>
<td>CFM</td>
<td>20%</td>
<td>0.54</td>
</tr>
<tr>
<td>Ply 7</td>
<td>Glass Veil</td>
<td>15%</td>
<td>0.35</td>
</tr>
</tbody>
</table>

**Figure 3** Typical test article and specimen geometry (specimen SL2L2T shown)

**Bearing Test Results**

A total of 13 bearing tests sets are reported on in this paper – based on a range of configurations as discussed previously. The specimen nomenclature includes the connection type (single lap “SL” or double lap “DL”), the number of fasteners in the joint, and the fastener configuration. A typical single-lap specimen and the geometry of the lap-splice is shown in Figure 3. This single lap specimen has four fasteners, two in the longitudinal direction and two in the transverse direction. The FRP is cut in the pultrusion (longitudinal direction) as are all specimen groups unless noted. A list of the bearing tests and the test results are given in Table 3. The bearing
test results are taken as the peak force observed during the test. In addition to the peak force, the force carried by each fastener per each ply of FRP is given for comparison. The failure modes observed in the given tests series are also provided.

Figure 4 shows the typical results from single-fastener bearing tests – including the self-tapping fastener and two bolts in pin-bearing representing the minor (4.0 mm) and major diameter (5.5 mm) of the self-tapping fastener. The bolted fastener data shows a set of localized “valleys” in the load-slip relationship. These valleys correspond to the onset of bearing failure at the surface of the FRP, due to out-of-plane rotation caused by the eccentricity inherent in single-lap shear tests. These valleys are either not present or difficult to detect in the self-tapping fastener data. The smaller 4.0 mm diameter test closely matches the bearing strength of the self-tapping fastener. It is clear however that the stiffness of the self-tapping connection is much higher than the stiffness of either of the bolted joints.

<table>
<thead>
<tr>
<th>Test</th>
<th>Strength (kN)</th>
<th>COV</th>
<th>Force per Screw (kN)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>SL1</td>
<td>5.22</td>
<td>6.2%</td>
<td>5.22</td>
<td>Bearing</td>
</tr>
<tr>
<td>SL2L</td>
<td>9.74</td>
<td>4.4%</td>
<td>4.87</td>
<td>Bearing</td>
</tr>
<tr>
<td>SL2T</td>
<td>8.92</td>
<td>6.0%</td>
<td>4.46</td>
<td>Bearing and fastener pull-through</td>
</tr>
<tr>
<td>SL3L</td>
<td>14.6</td>
<td>3.3%</td>
<td>4.83</td>
<td>Bearing</td>
</tr>
<tr>
<td>SL4L</td>
<td>19.1</td>
<td>4.5%</td>
<td>4.78</td>
<td>Bearing</td>
</tr>
<tr>
<td>SL2L2T</td>
<td>17.7</td>
<td>2.7%</td>
<td>4.43</td>
<td>Bearing and some block shear</td>
</tr>
<tr>
<td>SL2L - 45°</td>
<td>7.21</td>
<td>4.6%</td>
<td>3.61</td>
<td>Net tension</td>
</tr>
<tr>
<td>SL2L - 90°</td>
<td>6.21</td>
<td>4.7%</td>
<td>3.11</td>
<td>Net tension</td>
</tr>
<tr>
<td>SL1 Pin Bearing – Minor (4 mm) no fastener preload</td>
<td>4.93</td>
<td>10.7%</td>
<td>4.93</td>
<td>Bearing</td>
</tr>
<tr>
<td>SL1 Pin Bearing – Minor (4 mm) with fastener preload</td>
<td>6.11</td>
<td>10.0%</td>
<td>6.11</td>
<td>Bearing</td>
</tr>
<tr>
<td>SL1 Pin Bearing – Major (5.5 mm) no fastener preload</td>
<td>6.56</td>
<td>4.3%</td>
<td>6.56</td>
<td>Bearing</td>
</tr>
<tr>
<td>SL1 Pin Bearing – Major (5.5 mm) with fastener preload</td>
<td>7.51</td>
<td>4.1%</td>
<td>7.51</td>
<td>Bearing</td>
</tr>
</tbody>
</table>

This improved stiffness of the self-tapping joint is further illustrated in Figure 5, which shows the offset bearing strength of the self-tapping fastener, using the 2% offset from ASTM D5961 and the 4% offset from D953. It is
clear than the self-tapping fastener will have a higher offset bearing strength than either of the bolted connections. From the photograph, it is apparent that the self-tapping fastener demonstrates significant post-peak ductility.

The behavior of multi-fastener joints is similar to that of the single-fastener joints. For lap shear joints with 2, 3 and 4 screws in a line, the capacity increases linearly with the addition of fasteners. The 4 fastener connection shown in Figure 3 loses some capacity relative to the single-line joint, probably due to the mixture of bearing and block shear failures observed in the joint. The size of the joint needs to be increased slightly to preclude the block shear failure.

Figure 5 SL1 bearing test and offset bearing strength of self-tapping fastener joint

CONCLUSIONS

This research demonstrates the effectiveness of self-tapping fasteners as a method for joining thin-wall pultruded FRP members to steel. The techniques are applicable to achieving rapid field connections, especially in those instances where blind fastening, that is, the ability to access the back side of the connection, is required. Future publications from this research will explore the durability of the connections using these fasteners, and will demonstrate the design requirements for connections with loadings skew to the longitudinal axis of the FRP composite.

REFERENCES


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INFLUENCE OF USING PITCH-BASED ULTRA-HIGH MODULUS CARBON FIBRES ON THE STRUCTURAL BEHAVIOUR OF FRP REINFORCED CONCRETE SLABS

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1 Chair of Conceptual and Structural Design,
Technische Universität Berlin, Germany, Email: bernd.zwingmann@tu-berlin.de

ABSTRACT

FRP reinforcement bars made out of glass fibres, basalt fibres or high tenacity (HT) carbon fibres have lower Young’s modulus but higher strength when compared to steel bars. Thus, the deformation of FRP reinforced concrete slabs that are designed for Ultimate Limit State (ULS) will be much higher. Limiting deformation under service load requires additional reinforcement. Therefore, unwanted costs may arise through added material, labour and space for arrangement. Ultra-high modulus (UHM) carbon fibres reach a Young’s modulus of more than 900 GPa at a high tension strength of 3.7 GPa. CFRP reinforcement bars made out of these fibres have higher extension stiffness as steel bars despite the smaller cross section. The structural behaviour of eight reinforced concrete slabs is analysed in an experimental parameter study with varying reinforcement material and quantity. The results show that UHM carbon fibres can significantly reduce the deflection under service load without losing load bearing capacity. A promising alternative to CFRP reinforcement with HT carbon fibres is demonstrated by using pitch-based UHM carbon fibres instead of highly expensive PAN-based UHM carbon fibres.

KEYWORDS

Ultra-high modulus carbon fibre, pitch-based carbon fibre, FRP RC slab, experimental study.

INTRODUCTION

Fibre reinforced polymer bars (FRP bars) have a higher strength than steel and are resistant to corrosion. Thus, concrete elements reinforced with FRP bars are highly resistant and durable. The required amount of reinforcement is smaller for FRP than for steel reinforced concrete slabs that are designed for Ultimate Limit State (ULS). Due to this fact and the low Young’s modulus of the fibres the deflection of FRP reinforced slabs under service load is unacceptable high in comparison to steel reinforced slabs. To satisfy the Serviceability Limit State (SLS) additional reinforcement is needed. CFRP bars are commonly made of HT carbon fibres (HT bars). CFRP bars based on UHM carbon fibres (UHM bars) have much higher Young’s modulus. Using UHM bars, the deflection can be reduced significantly without increasing the amount of FRP reinforcement. Figure 1 shows the deformation of a reinforced concrete slab using different types of reinforcement.

Figure 1 Concrete slabs of same load-bearing capacity reinforced with steel, HT or UHM bars
Two series of experiments were carried out to analyse the deformation behaviour and load-bearing behaviour of concrete slabs reinforced with FRP. Four different materials were used as reinforcement: steel bars, GFRP bars, HT bars and UHM bars.

**METHOD OF SOLUTION**

**Pull-out-test**

Pultruded CFRP-bars have a smooth surface, which has to be treated to create bond between the bar and the concrete. One possibility is a sand coating glued with epoxy resin. Another possibility is milling the rod’s surface. Both bar surfaces are compared to choose the most favourable bond condition for the slabs. For ComBAR® (Schöck Bauteile GmbH 2013) the second method is used to create a spindle-shaped design. Alternatively the spindle can be shaped on the surface with epoxy glue. The different bond conditions were determined by testing a sand coated and a spindle-shaped CFRP bar in comparison to a ComBAR® and a steel bar. The spindle was shaped using viscous epoxy resin Sikadur®30-normal (Sika Deutschland GmbH 2013) and tape that was placed before and removed after applying the epoxy. Figure 2 shows the tested bar surfaces.

The performed pull-out-test was based on RILEM-RC 6 (RILEM 1983). To determine the bond between reinforcement and concrete a bar with a defined diameter (Ø) has to be concreted into a cube with a defined geometry and pulled out (see Figure 3).

The model code for concrete structures (CEB-FIB 2013) defines the mean bond-stress-slip relationship of ribbed bars (see black lines in Figure 4) to evaluate the bond-behaviour. The bond-stress $\tau_b$ is calculated dividing the pull-out-force by the bonding surface. The slip is recorded underneath the specimens using a displacement transducer. The linear ascending part refers to the elastic adhesive bond, which changes to the shear-bond (curved part). This is the shear stress between the concrete corbels and the ribs. At the highest point of the graph $\tau_{b,max}$ a sustained
plateau occurs. Then the concrete corbels fail and the branch descends. The following constant curve refers to the friction-bond $\tau_{b,f}$ between the ribs and the concrete.

![Figure 4 Maximum bond-stress-slip relationships for steel bar and FRP bars](image)

For each material three specimens were tested. The results for the specimens with the maximum bond-stress of each material are shown in Figure 4. The pull-out-tests show that the maximum bond-stress is reached by the steel bar with a mean value of 23.2 MPa and a slip of 1.08 mm. CFRP-sand surface and CFRP-spindle-shaped surface have the stiffest bond with a slip of 0.39 mm and 0.29 mm. CFRP-sand reaches a mean maximum bond-stress of 17.4 MPa, which is twice as much as the CFRP-spindle-shaped with 8.0 MPa. The friction-bond of CFRP-spindle-shaped and CFRP-sand are very close to each other. In Table 1 the mean values of the test results are shown. Based on the results CFRP-sand is chosen for the reinforcement of the investigated slabs.

<table>
<thead>
<tr>
<th>property</th>
<th>symbol</th>
<th>unit</th>
<th>steel</th>
<th>GFRP</th>
<th>CFRP-sand</th>
<th>CFRP-spindle-shaped</th>
</tr>
</thead>
<tbody>
<tr>
<td>maximum bond-stress</td>
<td>$\tau_{b,max}$</td>
<td>[MPa]</td>
<td>23.2</td>
<td>20.6</td>
<td>17.4</td>
<td>8.0</td>
</tr>
<tr>
<td>slip at maximum bond-stress</td>
<td>$s_{\tau bmax}$</td>
<td>[mm]</td>
<td>1.08</td>
<td>0.89</td>
<td>0.39</td>
<td>0.29</td>
</tr>
<tr>
<td>friction-bond</td>
<td>$\tau_{b,f}$</td>
<td>[MPa]</td>
<td>6.6</td>
<td>7.7</td>
<td>4.2</td>
<td>3.4</td>
</tr>
</tbody>
</table>

**Bending-test**

The concrete slab shown in Figure 5 had a length of 210 cm, a width of 30 cm and a height of 8 cm. The slab was supported like a single span beam with a span of $l_{eff} = 180$ cm. To perform a 4-point-bending test a traverse applied the load at two points. The force and displacement of the hydraulic cylinder was recorded. In addition load cells were installed below the traverse and three displacement transducers under the slab. Two additional displacement transducers recorded the strain on the front side to calculate the stress of the concrete and the reinforcement.

![Figure 5 Experimental setup for the 4-point-bending test](image)
Steel, GFRP bars, HT bars and UHM bars were used as reinforcement. The UHM bars were made out of continuous DIALEAD K13916 fibres (*Mitsubishi Plastics Inc.* 2015). They were laminated by hand, because commercially pultruded bars could not be obtained. The sand-coated UHM bar is shown in Figure 2. A concrete C50/60 was used with a maximum grain diameter of 16 mm. The properties of the materials are shown in Table 2.

<table>
<thead>
<tr>
<th>property</th>
<th>symbol</th>
<th>unit</th>
<th>steel</th>
<th>GFRP</th>
<th>HT</th>
<th>UHM</th>
</tr>
</thead>
<tbody>
<tr>
<td>material</td>
<td></td>
<td></td>
<td>B500A</td>
<td>ComBAR®</td>
<td>T 700</td>
<td>K13916</td>
</tr>
<tr>
<td>tensile strength</td>
<td>$f_\text{t}$</td>
<td>[MPa]</td>
<td>613.5$^1$</td>
<td>1378$^1$</td>
<td>2500$^3$</td>
<td>500$^4$</td>
</tr>
<tr>
<td>Young’s modulus</td>
<td>E</td>
<td>[MPa]</td>
<td>203447$^1$</td>
<td>63613$^1$</td>
<td>140000$^3$</td>
<td>172400$^4$</td>
</tr>
<tr>
<td>breaking strain</td>
<td>$\varepsilon_\text{u}$</td>
<td>[%]</td>
<td>30.02$^2$</td>
<td>21.66$^2$</td>
<td>17.86$^3$</td>
<td>2.90$^4$</td>
</tr>
<tr>
<td>fibre-volume</td>
<td>$\varphi$</td>
<td>[%]</td>
<td>-</td>
<td>75$^1$</td>
<td>63$^3$</td>
<td>21$^2$</td>
</tr>
<tr>
<td>fibre-tensile strength</td>
<td>$f_\text{ft}$</td>
<td>[GPa]</td>
<td>-</td>
<td>2.7-2.8</td>
<td>4.9$^3$</td>
<td>3.0</td>
</tr>
<tr>
<td>fibre-Young’s modulus</td>
<td>$f_\text{ft}$</td>
<td>[GPa]</td>
<td>-</td>
<td>81-83</td>
<td>230$^3$</td>
<td>760</td>
</tr>
</tbody>
</table>

$^1$Schöck Bauteile GmbH 2016  $^2$Calculated  $^3$vDijk Pultrusion Products 2014  $^4$Own test results

The concrete’s properties were obtained by material tests. The cylindrical compressive strength $f_\text{cm}$ was 61.9 MPa at a strain of 3.99 %. The tensile strength $f_\text{ctm}$ and the Young’s modulus $E_\text{cm}$ were 3.4 MPa and 29 GPa.

Two series of experiments were carried out to study the deformation behaviour and load-bearing behaviour. In the first test series (ULS<sub>eq</sub>), the slabs were designed for the same load capacity so that failure occurs at the same maximum load. The second series (SLS<sub>eq</sub>) was designed to have the same deflection at a defined load. The deflection $w$ should not exceed the limit $l_{\text{eff}}/200$. Therefore the limit deflection was designed to be between 8 and 8.4 mm for a bending moment of 1.46 kNm. The quantity of required reinforcement for both series is listed in Table 3 and 4.

<table>
<thead>
<tr>
<th>property</th>
<th>symbol</th>
<th>unit</th>
<th>steel</th>
<th>GFRP</th>
<th>HT</th>
<th>UHM</th>
</tr>
</thead>
<tbody>
<tr>
<td>reinforcement diameter</td>
<td>$\varnothing$</td>
<td>[mm]</td>
<td>6</td>
<td>8</td>
<td>4</td>
<td>7.7$^4$</td>
</tr>
<tr>
<td>number of reinforcement bars</td>
<td>n</td>
<td>[MPa]</td>
<td>4</td>
<td>2</td>
<td>2</td>
<td>5</td>
</tr>
<tr>
<td>reinforcement area</td>
<td>A</td>
<td>[cm²]</td>
<td>1.13</td>
<td>1.01</td>
<td>0.25</td>
<td>2.33</td>
</tr>
<tr>
<td>fibre area</td>
<td>$A_\text{f}$</td>
<td>[cm²]</td>
<td>1.13</td>
<td>0.76</td>
<td>0.16</td>
<td>0.49</td>
</tr>
<tr>
<td>ultimate moment</td>
<td>$M_\text{u}$</td>
<td>[kNm]</td>
<td>2.87</td>
<td>2.59</td>
<td>2.87</td>
<td>2.94</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>property</th>
<th>symbol</th>
<th>unit</th>
<th>steel</th>
<th>GFRP</th>
<th>HT</th>
<th>UHM</th>
</tr>
</thead>
<tbody>
<tr>
<td>reinforcement diameter</td>
<td>$\varnothing$</td>
<td>[mm]</td>
<td>6</td>
<td>8</td>
<td>4</td>
<td>7.7$^4$</td>
</tr>
<tr>
<td>number of reinforcement bars</td>
<td>n</td>
<td>[MPa]</td>
<td>2</td>
<td>4</td>
<td>6</td>
<td>2</td>
</tr>
<tr>
<td>reinforcement area</td>
<td>A</td>
<td>[cm²]</td>
<td>0.57</td>
<td>2.01</td>
<td>0.75</td>
<td>0.93</td>
</tr>
<tr>
<td>fibre area</td>
<td>$A_\text{f}$</td>
<td>[cm²]</td>
<td>0.57</td>
<td>1.51</td>
<td>0.47</td>
<td>0.20</td>
</tr>
<tr>
<td>moment for same deflection</td>
<td>$M_\text{eq}$</td>
<td>[kNm]</td>
<td>1.46</td>
<td>1.46</td>
<td>1.46</td>
<td>1.46</td>
</tr>
<tr>
<td>deflection</td>
<td>w</td>
<td>[mm]</td>
<td>8.4</td>
<td>8.0</td>
<td>8.3</td>
<td>8.3</td>
</tr>
</tbody>
</table>

**RESULTS AND DISCUSSIONS**

The moment-deflection-curves for all specimens of ULS<sub>eq</sub> are shown in Figure 6. Using guaranteed strength and safety factors instead of mean strength of the bars by mistake the ultimate moments were calculated wrongly (compare $M_\text{u}$ in Table 3 and 5). Nevertheless, the results of ULS<sub>eq</sub> show how UHM-fibres reduce deflection. Table 5 shows the ultimate moment and deflection of FRP reinforced slabs in comparison to steel reinforced slab. Values in prentices show the difference to the steel reinforced slab. The results show that for a similar ultimate moment ($\pm 20 \%$) the deflection for HT bars is much higher than for steel bars. In contrast the deflection is much lower for UHM bars. GFRP bars cause extensive deflection despite their highest ultimate moment. The fibre area of UHM bars is 3 times bigger but the deflection is almost 6 times lower compared with HT bars. None of the FRP bars provides ductility.
Table 5 Bending-test results for $ULS_{eq}$

<table>
<thead>
<tr>
<th>property</th>
<th>symbol</th>
<th>unit</th>
<th>steel</th>
<th>GFRP</th>
<th>HT</th>
<th>UHM</th>
</tr>
</thead>
<tbody>
<tr>
<td>fibre area</td>
<td>$A_f$</td>
<td>[cm$^2$]</td>
<td>1.13</td>
<td>0.76</td>
<td>0.16</td>
<td>0.49</td>
</tr>
<tr>
<td>ultimate moment</td>
<td>$M_u$</td>
<td>[kNm]</td>
<td>4.0</td>
<td>7.6 (+90%)</td>
<td>3.2 (-20%)</td>
<td>4.8 (+20%)</td>
</tr>
<tr>
<td>ultimate deflection</td>
<td>$w_u$</td>
<td>[mm]</td>
<td>48</td>
<td>127.3 (+165%)</td>
<td>94.5 (+97%)</td>
<td>16 (-67%)</td>
</tr>
<tr>
<td>mode of failure</td>
<td></td>
<td></td>
<td>bar</td>
<td>concrete</td>
<td>bond</td>
<td>bar</td>
</tr>
</tbody>
</table>

In Figure 6 the moment-deflection-diagram for $ULS_{eq}$.

In Figure 7 the concrete slabs of $ULS_{eq}$ are shown during the test with the same moment ($M = 2.57$ kNm). The deflections of slabs reinforced with GFRP bars and HT bars are much higher than those of steel bar and UHM bar reinforced slabs. In addition to low deflection the cracks are evenly distributed with small widths when using UHM bars. This is caused by the stiff bond (see Figure 4) and the high Young’s modulus.

Figure 6 Moment-deflection-diagram for $ULS_{eq}$

Figure 7 Deflection of the $ULS_{eq}$-slabs with the same bending moment
Test results for SLS$_{eq}$ are shown in Figure 8 and Table 6. The slabs with the four different reinforcement materials show similar deformation behaviour such as planned for SLS$_{eq}$ (scaled up view in Figure 8). For a moment of 1.46 kNm the slabs deflect approximately 5 mm (see Table 6) which is lower than the calculated value of 8.8 mm. UHM bars require less than 50 % of HT fibre cross section and 13 % of glass fibre cross section for the same deformation. The required amount of GFRP and HT reinforcement to achieve low deformation leads to an unnecessary high ultimate moment. An adequate balance between deformation and ultimate moment is achieved by the usage of UHM bars. The load-bearing behaviour is similar to the steel reinforced slab.

<table>
<thead>
<tr>
<th>property</th>
<th>symbol</th>
<th>unit</th>
<th>steel</th>
<th>GFRP</th>
<th>HT</th>
<th>UHM</th>
</tr>
</thead>
<tbody>
<tr>
<td>fibre area</td>
<td>$A_f$</td>
<td>[cm²]</td>
<td>0.57</td>
<td>1.51</td>
<td>0.47</td>
<td>0.2</td>
</tr>
<tr>
<td>deflection at 1.46 kNm</td>
<td>$w$</td>
<td>[mm]</td>
<td>4.9</td>
<td>4.3</td>
<td>5</td>
<td>5.5</td>
</tr>
<tr>
<td>ultimate moment</td>
<td>$M_u$</td>
<td>[kNm]</td>
<td>2.1</td>
<td>11.3</td>
<td>11.1</td>
<td>2.1</td>
</tr>
<tr>
<td>ultimate deflection</td>
<td>$w_u$</td>
<td>[mm]</td>
<td>28.6</td>
<td>109.5</td>
<td>106</td>
<td>10.7</td>
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<tr>
<td>mode of failure</td>
<td></td>
<td></td>
<td>bar</td>
<td>concrete</td>
<td>bond</td>
<td>bar</td>
</tr>
</tbody>
</table>

Figure 8 Moment-deflection-diagram for SLS$_{eq}$

CONCLUSIONS

This paper shows that CFRP-bars made out of UHM carbon fibres can significantly reduce the deformation of concrete slabs in comparison to other FRP bars. Less reinforcement can be used and unnecessary high ultimate moments can be avoided. Pitch-based carbon fibres can be an advantageous alternative for the highly expensive PAN-based carbon fibres to produce UHM bars.

REFERENCES

DEVELOPMENT OF BRAIDED BASALT FRP REBAR FOR REINFORCEMENT OF CONCRETE STRUCTURES

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2 Burgmann Packings Ltd, Dublin, Ireland.

ABSTRACT

In recent years, the development and use of Fibre Reinforced Polymer composite materials in infrastructure have gained increasing attention worldwide. More specifically, natural mineral fibres such as basalt are currently being developed and are showing promising properties. Within an appropriate polymer matrix, their use as reinforcement in concrete structures offers performance benefits related to their environmentally friendly and non-corrosible nature. In particular, BFRPs have the potential to replace conventional internal steel rebar and thus, to be the next generation material in concrete reinforcement applications. A detailed literature review indicates that a careful selection of the appropriate manufacture technique and design methodology are required in order to prevent brittle failure on a concrete structure reinforced with FRP composite material. This paper reports on how to use the additional helical reinforcement and the braid configuration in order to increase strength, structural ductility and long term durability. Moreover, this study outlines the development of an analytical numerical model to predict the longitudinal elastic modulus of braided composites, as well as its validation by comparison of the results with available data from the literature.

KEYWORDS

Fibre Reinforced Polymer (FRP), basalt fibre, braiding technique, infrastructure, concrete reinforcement, BFRP rebars.

INTRODUCTION

The long-term durability of steel reinforced concrete (RC) structures, subjected to repeated loading and aggressive environmental factors, has become a major concern in the civil engineering construction industry. Degradation of both mechanical properties and durability performance due to corrosion severely affects service lives and the safety of steel RC elements. This in turn raises the need for their effective rehabilitation (Hollaway 2010; Afifi et al. 2015; Fiore et al. 2015; Elgabbas et al. 2015). It is noteworthy that maintenance costs related to steel reinforcement corrosion are significant worldwide, with an estimated repair cost of over $12.8 billion per annum for bridges in the United States alone (Behnam and Eamon 2013). The use of fibre reinforced polymer (FRP) composite materials as a potential replacement for traditional steel in internal concrete reinforcement, has recently received a great deal of attention by civil engineering scientists, as well as industry (Hollaway 2010; Afifi et al. 2015). The advantages of FRP composite materials over steel include excellent corrosion resistance, good fatigue properties, damage tolerance, low specific gravity, non-magnetic properties, light weight, low energy consumption during fabrication, and the potential real-time monitoring of their behaviour (Portnov et al. 2013; Pastore and Ko 1999). In addition, these innovative materials are considered to offer an overall more economical option for construction applications (Fiore et al. 2015; Behnam and Eamon 2013). Summarising, the optimum development of a reinforced concrete type in which the usual internal steel rebars are replaced by FRP composite materials is consider to be an ongoing research topic worldwide, but the structural safety associated with this composite material is still not fully understood (Afifi et al. 2015; Elgabbas et al. 2015; Wang and Belarbi 2010).

The aim of this work is to design and develop novel BFRP rebars for manufacture using a braiding technique. They consist of basalt fibres and a thermosetting polymer matrix (epoxy resin), and the relation between braiding parameters and rebar properties is assessed. Theoretical calculations by means of numerical simulation are also presented to determine the stiffness properties of manufactured braided BFRP composites; model data is then compared with results available from the literature. It should be noted that these innovative rebars give a unique surface texture for better interfacial interactions with concrete; this feature will be further investigated in the next phase of the research.
LITERATURE REVIEW

Fibre Reinforced Polymer for concrete reinforcement

According to the literature, FRP composites consist of high tensile fibres (e.g. carbon, glass, basalt) embedded in polymer matrices (thermosetting or thermoplastic) leading to improved mechanical properties of the system (Fiore et al. 2015; Behnam and Eamon 2013). In particular, basalt fibre, an inorganic, natural fibre made from molten basalt rock at diameters ranging from 9 to 20 μm, has the potential to become a promising reinforcing material for FRP composites in infrastructure applications. This is due to its excellent properties, such as high tensile modulus, good alkali resistance and environmentally friendly origins (Ahmadi et al. 2012; Parnas et al. 2007; Banibayat and Patnaik 2014). It is broadly accepted that FRPs can be a viable alternative for replacing steel as internal concrete reinforcement in a cost effective manner, as these materials are competitive in terms of life-cycle costs (Whitehead and Ibell 2005). In addition, they have several performance benefits namely the corrosion resistance, high tensile strength, fatigue resistance and many more. However, a direct substitution between FRP and steel reinforcing bars is not possible due to various differences in their physical and mechanical properties. The linear stress-strain behaviour up to failure, the lack of ductility and energy absorbing capabilities are the main limitations that prevent the use of FRP rebars on a large scale (Portnov et al. 2013; Pastore and Ko 1999; Whitehead and Ibell 2005; Ibell et al. 2009). If FRP reinforcement is to be used successfully and widely in concrete structures, it must be used to its full potential (Ashour and Habeeb 2008; Whitehead and Ibell 2005). When FRP bars are simply placed in concrete as a direct substitute for steel bars, it is impossible to achieve this aim, as the concrete tensile strain capacity is low compared with the high capacity of FRP (Portnov et al. 2013). Therefore, it is clearly understood that, only through the rational and efficient use of FRP composite materials that can meet strength, ductility, stiffness, as well as reliability and cost demands, both construction industry and engineers, as well as manufacturers of these products, will gain the greatest benefit (Behnam and Eamon 2013; Wang and Belarbi 2010; Ibell et al. 2009).

Due to the growing interest on the use of FRP composites in construction, a small number of design guidelines, codes and specifications for their efficient use in civil engineering applications were developed during the last decade. Special publications by ACI - the ACI Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars, ACI-440.1R – and various other international standards, including the Japan Society of Civil Engineers, the British Standards Institution, the Canadian Standards Association International - CSA-S806-02 - that directly address the use of FRP, are available so far. However, the use of FRP for reinforcement instead of steel is far from widely accepted in the conservative construction industry, mostly because of the lack of familiarity with this material, the high initial costs and the lack of structural ductility. Moreover, design codes are slowly developing for FRP rebars compared to the well-established ones for the use of steel bars in concrete members (Afifi et al. 2015; Fiore et al. 2015; Elgabbas et al. 2015; Behnam and Eamon 2013).

To date most of the FRP reinforcement used in concrete has been manufactured by pultrusion, a low cost method providing composites with a constant cross section and a smooth surface (Fiore et al. 2015; Ahmadi et al. 2009). Recently there has been increasing interest among researchers in braiding techniques for FRP composite fabrication, aiming for improved mechanical properties and multiaxial orientation (Pereira et al. 2008, Tang and Postle 2001). The basic principle of the braiding technique is the interlacing of yarns in a way that they cross each other in a diagonal direction (Pastore and Ko 1999). Braiding angle – the angle between braid yarns and axis of braid structure- is an important parameter that affects mechanical properties of braids and is usually adjusted by changing the take-up speed in braiding machine, obtaining a range between 20 and 85° (Ahmadi et al. 2009; Branscomb et al. 2013). It should be noted that braided FRP rebars could obtain higher flexural & shear strength, increased durability and improved crack resistance, compared to pultruded ones that exhibit low bending strength, mostly due to uni-directional structure. Although the braiding method is less common as it is more expensive, it does present significant advantages in special civil engineering applications, mostly relating to the additional ductility provided and the increased bond between FRP and concrete, which has a direct influence on both the serviceability and the ultimate load-carrying capacity of the structure (Pereira et al. 2008; Ashour and Habeeb 2008).

EXPERIMENTAL WORK

In this paper, a theoretical approach for the prediction of the elastic modulus of braided BFRP rebars is established, based on numerical simulation and mathematical modelling after analyzing the manufacture of the braided preforms. Comparisons are made between the values predicted from the present model and the literature data. In addition, a braided BFRP helix configuration for internal concrete reinforcement is attempted to be produced with epoxy resin impregnation.
Manufacturing design and process

The goal of this research activity is to develop an understanding of the mechanical behaviour of braided composite bars and correlate it with textile processing conditions. An important challenge, that needs special attention when designing textile composite materials, is the complex nature of the processing parameters, such as yarn size, number of carriers in the braiding machine, braid angle, in order to ensure optimum behaviour of the final product and obtain the desired physical and mechanical properties for its end-use. The development of braided BFRP preforms for use as rebars in concrete reinforcement after proper impregnation in epoxy resin and the influence of the braiding angle and the number of braiding layers are the main fields investigated within this study. The objective of these trials is the production of braided BFRP rebars, consisting of a braided core (~ 3mm) and surrounded by two layers of PET material (Figure 1). These are introduced to promote resin flow, and an outer finishing braiding layer is added to complete the required dimensions. Basalt fibres are the reinforcing material and the matrix is epoxy resin. The target outer diameter of samples is 5.0 mm (Figure 2). Table 1 exhibits in detail the fundamental material properties of basalt, PET fibres and epoxy resin used on the trials.

Figure 1 Basalt core – PET mesh for resin impregnation
Figure 2 Cross-section of braided BFRP rebar

Table 1 Material properties of BFRP composites

<table>
<thead>
<tr>
<th>Property</th>
<th>Basalt</th>
<th>PET Mono</th>
<th>Epoxy resin</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (g/cm³)</td>
<td>2.7</td>
<td>1.39</td>
<td>1.15</td>
</tr>
<tr>
<td>Elastic modulus (GPa)</td>
<td>87/ 89</td>
<td>2.65</td>
<td>10</td>
</tr>
<tr>
<td>Linear density (TEX)</td>
<td>300/ 600</td>
<td>97</td>
<td></td>
</tr>
<tr>
<td>Yarn diameter (mm)</td>
<td>0.38/ 0.53</td>
<td>0.3</td>
<td></td>
</tr>
</tbody>
</table>

*Note:* Yarn diameter was calculated using the following equation: \( D = \sqrt{\frac{4 \times \text{tex} \times 10^5}{\rho \pi}} \)

Numerical analysis

Simplified theoretical approaches for FRPs often presume a layup of only unidirectional reinforced layers and homogenization approaches for the prediction of structural properties. However, different kinds of fabrics and manufacture techniques are often applied for the layup of FRP materials (Tang and Postle 2001). Several studies report the use of classical lamination theory (CLT) approach for the analysis of braided FRP laminated structure, and these formed the basis for this work (Valentino et al. 2013; Srikanth and Rao 2014; Bank 2006).

For each layer of every BFRP sample, the four independent engineering parameters affecting their stiffness behaviour (longitudinal/ \( E_x \), transverse/ \( E_y \), shear/ \( E_s \), modulus and Poisson’s ratio/ \( \nu \)) were estimated using the following equations:

\[
E_x = \varphi_f E_f + (1 - \varphi_f) E_m
\]

\[
E_y = \frac{E_x \left(1 - \sqrt{\varphi_f(1-\varphi_f)}\right)}{E_m}\nu_x \nu_y E_x
\]

\[
E_s = \frac{E_x}{2(1+\nu)}
\]

\[
\nu_x E_y = \nu_y E_x
\]

where \( \nu_x \) and \( \nu_y \) are the major and minor Poisson ratio respectively, \( E_f \) and \( E_m \) are the Young’s modulus of fibre and matrix respectively and \( \varphi_f \) is the fibre volume content given by Eq. 5.

\[
\varphi_f = \frac{U_f}{U_c}
\]

where \( U_f \) and \( U_c \) denote the volume of fibre and composite respectively. The fibre volume content is an indicator for the achieved quality and serves as a base for the prediction of braid’s mechanical properties.
Then, the on-axis stiffness matrix \([Q_{ij}]\) for each braid layer is calculated, defining the six off-axis stiffness coefficients along with braid angle. The stiffness matrix \([A–B–D]\) and the cofactor expansion matrix \([C]\) for each BFRP sample can be obtained from terms \(A_{ij}\), \(B_{ij}\), \(D_{ij}\), expressing the extensional, flexural and extensional-bending coupled response respectively, as follows.

\[
A_{ij} = \sum_{k=1}^{n} Q_{ij}^{k} h_{k}
\]

(6)

\[
B_{ij} = \sum_{k=1}^{n} Q_{ij}^{k} (-z_{k} h_{k})
\]

(7)

\[
D_{ij} = \sum_{k=1}^{n} Q_{ij}^{k} (h_{k}z_{k}^2 + \frac{h_{k}^3}{12})
\]

(8)

where \(z_{k}\) is the midplane distance for each layer \(k\) and \(n\) the total number of layers in each sample.

Finally, Eq. 9 forms the basis for the evaluation of the elastic properties of braided BFRP rebars.

\[
E_{x_{FRP}} = \frac{DET([A–B–D])] / DET([C])}
\]

(9)

RESULTS AND DISCUSSIONS

Braided BFRP rebar preforms were produced varying the number of layers and the parameters of braiding process to achieve the desired structural geometry and meet the performance characteristics of existing rebar reinforcement. More specifically, the production of the core focused on two configurations: a single layer of 600 tex Basalt and a 2-layer version using 300 tex Basalt. Two approaches were also used for the PET flow mesh. The first approach was the production of a sleeve using 32 threads, whereas the second was over braiding of the core using 24 threads. The outer layers were produced using 600 tex Basalt with the target braid angle of ±45° (Table 2). A complete braided sample, made in Burgmann Packings Ireland, is shown in Figure 3.

![Figure 3 Braided BFRP rebar preform sample.](image)

![Figure 4 Resin impregnated braided BFRP rebar.](image)

<table>
<thead>
<tr>
<th>Table 2 Input data for BFRP composite analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>Layer</td>
</tr>
<tr>
<td>-------</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
<tr>
<td>5</td>
</tr>
</tbody>
</table>

Note: All values are an average of readings taken at several locations of manufactured BFRP samples; OD: Outer diameter (mm); Braid yarns: Basalt (tex), PET Monofilament; Angle (°).

The three selected braided preforms were then tested in SuperTEX Austria to evaluate their suitability for epoxy resin impregnation in a helical configuration. Each sample was found to give similar infusion results, with sample n°3 observed to exhibit the best behaviour (Figure 4).

The effective longitudinal in-plane modulus \(E_{x_{FRP}}\) was also calculated using CLT and Equations 1 – 9. This approach produced theoretical values of 25.80, 27.21 and 27.54 GPa for BFRP samples 1, 2, 3 respectively. The validity of this numerical investigation was assessed by a direct comparison with reference values of pultruded BFRP rebars from the literature (Elgabbas et al. 2015; High et al. 2015; Hui et al. 2012). This showed that the numerically determined elastic moduli using the CLT approach are largely similar, with BFRP 3 performing slightly better. Although the modulus values are relatively low, it should be noted that the small diameter braiding
process leads to relatively low fibre volume fractions. The limitation in braiding carriers in order to meet the diameter demands for helical configuration is considered to be the main factor that prevents higher rates compared to BFRP rebars produced by pultrusion technology, which can obtain experimental values of modulus around 40 – 50 GPa according to the literature. This can be addressed by using larger diameter samples, with higher fibre contents in the outer layers. It should also be noted that the braiding technique can offer significant performance benefits in specific structural applications in which additional ductility or/ and increased bond between FRP and concrete is needed.

For the next stage of this research experimental tests regarding physical and mechanical properties of braided BFRP rebars will take place at University College Dublin. These aim to verify the validity of the CLT numerical approach for accurate prediction of the properties of three-dimensional braided composites.

**CONCLUSIONS**

This paper demonstrates research that is currently in progress concerning the development of braided BFRP bars for concrete reinforcement. The described preliminary work has been conducted to understand the behaviour of braided fabrics and the influence of geometrical factors and processing conditions on physical & mechanical properties of braided rebars. The aim of this study is to successfully produce a braided BFRP helix for internal concrete reinforcement and to evaluate the stiffness of this braided composite using CLT approach. In particular, the mechanical response and the stiffness of braided BFRP composites have been estimated with numerical analysis and, in a further step, this theoretical model was validated using reference data showing a good agreement. The findings of this work include a useful tool for the design of BFRP rebars manufactured using braiding, capable of being applied to other braided geometries. Areas of future work within this project include the detailed investigation of FRP influence on structural reliability and an in-depth research on their strength, response and durability in order to obtain an optimized design and determine how this promising material can contribute to a more durable infrastructure.

**ACKNOWLEDGMENTS**

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AXIAL BEHAVIOUR OF CIRCULAR GEOPOLYMER CONCRETE COLUMNS
REINFORCED WITH GFRP BARS AND SPIRALS

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Email: ginghis.maranan@usq.edu.au
2 Department of Civil Engineering, University of Sherbrooke, Canada
3 Department of Infrastructure Engineering,
The University of Melbourne, Australia.

ABSTRACT

This paper presents the results of an experimental investigation of the axial behaviour of circular geopolymer concrete columns longitudinally and transversely reinforced with glass fibre reinforced polymer (GFRP) bars and spirals, respectively. Three full-scale short columns with a slenderness ratio \( L/r \) of 8 were fabricated and tested: one unconfined column and two confined columns with spiral pitch \( s \) of 50 mm and 100 mm on-centers. The GFRP spirals have a nominal diameter, tensile strength, and tensile modulus of 9.5 mm, 1184 MPa, and 62.6 GPa, respectively. Based on the experimental results, the GFRP spirals enhanced the compression performance of the unconfined column. The well-confined column \( s=50 \text{ mm} \), yielded the largest load-carrying and deformation capacities, approximately 22\% and 174\%, respectively, higher than that of the unconfined column. This column also showed better compression performance such as less brittle failure, higher ductility (67\%), and greater confinement efficiency (28\%) compared to the column with wider spiral pitch \( s=100 \text{ mm} \). Finally, the tested columns showed relatively superior ductility and confinement efficiency compared to the conventional concrete columns reinforced with GFRP bars and spirals.

KEYWORDS

Geopolymer concrete columns, GFRP bars, GFRP spirals, ductility, confinement efficiency.

INTRODUCTION

Glass fibre-reinforced polymer (GFRP) bar and geopolymer concrete are two emerging building materials that are normally substituted for steel bar and ordinary concrete, respectively, to deal with the corrosion-induced durability problems and cement sustainability issues, respectively, of the conventional reinforced concrete (RC) system. Aside from being corrosion resistant, GFRP bars have high tensile strength, lightweight, electromagnetic neutral, and non-conductive, with the added benefits of ease of transportation and fabrication. Geopolymer concrete, on the other hand, is a sustainable construction material because its binder, the geopolymer, can be manufactured using alkali-activated silica- and alumina-rich industrial waste materials and its production process consumes low energy and emits low CO\(_2\) in the atmosphere. It is also resistant to elevated temperatures, highly stable in aggressive environments, and can develop high mechanical strength at a short period.

Several studies reported that both the GFRP bars and geopolymer concrete were suitable for the fabrication of compression members. The study conducted by Tobbi et al. (2012) showed that the GFRP bars contributed 10\% of the column capacity, which was practically analogous to steel’s contribution (12\%). Furthermore, the more recent experimental investigation done by Afifi (2013) and Mohamed et al. (2014) showed that the compression performance of circular columns reinforced with GFRP bars and spirals was similar to that of the traditional RC columns. They also reported that the GFRP spirals effectively enhanced the ductility of the columns, provided good confinement of the concrete core, and provided sufficient restraint against buckling of longitudinal bars. On the other hand, the research works done by Sumajouw and Rangan (2006) and Sujatha et al. (2012) revealed that the crack pattern, failure mode, and compression behaviour of geopolymer concrete columns were similar to that of the ordinary-concrete columns. It was also reported that the design procedures (Sumajouw and Rangan 2006) and analytical equations (Sarker 2009) for the typical RC column were also applicable to steel-reinforced geopolymer concrete (S-RGC) columns. With the stated advantageous characteristics of the GFRP bars and geopolymer concrete, combining them would offer a promising technology, herein called GFRP-reinforced
geopolymer concrete (GFRP-RGC), in fabricating new compression members with high durability, high sustainability, and adequate structural integrity. In this study, the axial behaviour of geopolymer concrete columns reinforced with GFRP bars and spirals was investigated. The compression contribution of longitudinal GFRP bars was quantified, the effects of the presence and pitch of spirals were evaluated, and the overall behaviour of GFRP-RGC column was compared to that of the GFRP-reinforced concrete (GFRP-RC) columns.

EXPERIMENTAL PROGRAM

Materials & Test Specimens

High modulus sand-coated GFRP bars and spirals with nominal diameters of 15.9 mm and 9.5 mm, respectively, were used as longitudinal and transverse reinforcement (Figure 1). The bars were manufactured through a pultrusion process wherein the E-glass fibres are impregnated with a modified vinyl ester resin. The properties of the bars, as specified by the manufacturer, were summarised in Table 1. A patented geopolymer concrete made up of 10-20 mm coarse aggregates, fine and medium sands, water, plasticizer, and a geopolymer binder made up of alkali-activated fly ash and slag was used in the study. The unique feature of this geopolymer concrete is that it can be cured in ambient conditions and it has a low heat of hydration, approximately 15°C (Aldred n.d.), which is just 12% of the glass transition temperature (T_g) of GFRP bars. Following the ASTM C39/C39M-15a (2015), the average compressive strength (f'_c) and elastic modulus (E_c) of the geopolymer concrete were 38 MPa and 33 GPa, respectively. Three full-scale short columns with a slenderness ratio (L/r) of 8 were cast and tested. The diameter (D) and height (L) of the column were 250 mm and 1000 mm, respectively. Each column was longitudinally reinforced with six 15.9 mm GFRP bars, yielding a reinforcement ratio (\(\rho_f\)) of 2.43%. The first column, the control specimen, was fabricated without transverse reinforcement in the tested region. The second and third columns were transversely reinforced with GFRP spirals pitched (s) at 50 mm and 100 mm on-centers, respectively, which translate to transverse reinforcement ratios (\(\rho_{ft}\)) of 3.13% and 1.57%, respectively. The specimens were labelled as follows: GGC-s. The first three letters (GGC) stand for “GFRP-reinforced geopolymer concrete column” followed by the corresponding spiral pitch (s). Figure 2 shows the details and configurations of the tested columns.

![GFRP reinforcement](image)

(a) 15.9 mm bars  (b) 9.5 mm spirals

Figure 1 GFRP reinforcement

Table 1 Properties of GFRP reinforcement

<table>
<thead>
<tr>
<th>Øf, mm</th>
<th>Ab, mm²</th>
<th>fuf, MPa</th>
<th>Ef, GPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.5</td>
<td>71.3</td>
<td>1372</td>
<td>65.1 ± 2.5</td>
</tr>
<tr>
<td>15.9</td>
<td>197.9</td>
<td>1184</td>
<td>62.6 ± 2.5</td>
</tr>
</tbody>
</table>

Where: \(\text{Øf} =\) nominal diameter; \(Ab = \) nominal cross-sectional area; \(f_{uf} = \) guaranteed tensile strength (average value – 3x standard deviation); \(E_f = \) tensile modulus

Methods

Figure 3 shows the test setup employed in the study. The columns were confined at both ends with two pairs of 10 mm thick steel clamps to prevent end crushing and to ensure that failure occurs at the test region. In addition, 3 mm thick neoprene rubber were provided between the specimen and the support plates to ensure uniform load distribution across the cross section. Chicken wire was wrapped around the column specimens for safety purposes. The monotonically increasing axial loads were applied in displacement control mode using a hydraulic jack to allow for the observation of both the pre- and post-peak behavior. The magnitude of the applied loads was measured with a 3000 kN capacity load cell, whereas the corresponding deformations were measured with a string pot. The location of the electrical strain gauges are shown in Figure 2 depicts. Three strain gauges were mounted onto longitudinal bars and another three were attached to the geopolymer concrete surface, aligned with the bars. Four strain gauges set 90° apart were also used to capture the strains in the transverse reinforcement. The strain, load, deformation, and deflection readings were recorded using a data logger attached to the machine, while the failure modes were documented with a video recorder.
RESULTS AND DISCUSSION

Load-Deformation Relationship

Figure 4 shows the load-deformation response of the tested columns. The load-deformation curves of the unconfined and confined columns consisted of a relatively similar linear ascending segment, with an average stiffness of 311 kN/mm. This can be expected since, at this stage, the columns’ behavior was governed predominantly by the geopolymer concrete’s compressive properties with little or no significant contribution from the GFRP ties. After exceeding the peak load level, GGC-00 failed suddenly and did not show any post-peak behavior. This load represents the gross capacity of the unconfined geopolymer concrete column ($P_g$). GGC-50 and GGC-100, however, continued to carry additional loads, owing to the still-intact concrete core confined by spirals. The confined columns yielded two peak loads. The initial peak load embodies the $P_g$ of the confined column that marks the beginning of concrete cover spalling. The second peak, on the other hand, corresponds to the maximum load capacity of the confined geopolymer-concrete core ($P_c$) that signifies the initiation of geopolymer-concrete core crushing failure.

Failure mode

Figures 5 shows the post-failure overview of the tested columns. As can be expected, the specimens exhibited a compression-type of failure, suggesting the effectiveness of the design and construction procedure employed in the study. The simultaneous crushing of the geopolymer concrete and global buckling of the GFRP bars governed the failure of GGC-00. A well-formed cone on both ends (Figures 5a) characterized GGC-00’s post-failure configuration. GGC-50 (Figure 5b) and GGC-S100 (Figure 5c), on the other hand, failed in a relatively ductile and more complex manner compared to the control specimen and characterized by the formation of an inclined failure plane, depicted by red lines. The failure can be described as the sequential occurrence of the following mechanism: local buckling and compression rupture of GFRP-bars, geopolymer-concrete core crushing, and rupture of the GFRP spirals, specifically at the intersection of the longitudinal and transverse reinforcement (Figure 6).

Table 2 summarizes the $P_g$ and $P_c$ of the tested columns. The compression contribution of the unconfined geopolymer concrete was determined by subtracting the GFRP bars’ contribution ($P_{gb}$) from GGC-00’s $P_g$ (1772 kN) and then dividing the remaining load by the difference between the geopolymer concrete gross area ($A_e$) and the total bar area ($A_b$), ($P_g-P_{gb}$)/($A_e-A_b$). The result was equivalent to 34.42 MPa, which was approximately 90% of the average compressive strength of the standard geopolymer concrete cylinders used in the study. Interestingly, this ratio was higher than the commonly used value of 85% for estimating the theoretical capacity of ordinary-concrete column sections, which tends to support Maranan et al.’s (2015) conclusion that geopolymer concrete has better mechanical properties than ordinary concrete of the same grade such as higher elastic modulus (leading to its better compatibility with GFRP bars compared to normal concrete), greater ultimate compressive strain (as much as 4800 με), and larger tensile strength. The use of spirals improved the $P_g$ of GGC-00. The $P_g$ of GGC-50 (1838 kN) and GGC-100 (2063 kN) increased by 4% and 16%, respectively, owing to the activation of the lateral confining pressures of the spirals. The $P_c$ of GGC-50 was lower than that of GGC-100, which could be due to the presence of closely spaced ties that caused discontinuity of the geopolymer concrete between the cover and core, making the column more susceptible to early concrete-cover spalling. Only GGC-50 and GGC-100 yielded $P_c$. 

<table>
<thead>
<tr>
<th>Column</th>
<th>$P_g$ (kN)</th>
<th>$P_c$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GGC-00</td>
<td>1772</td>
<td>1563</td>
</tr>
<tr>
<td>GGC-50</td>
<td>1838</td>
<td>1897</td>
</tr>
<tr>
<td>GGC-100</td>
<td>2063</td>
<td>2153</td>
</tr>
</tbody>
</table>
equivalent to 2160 kN, and 1717 kN, respectively, which were 118% and 82% of their respective \( P_c \). As can be expected, the well-confined column (GGC-50) yielded \( P_c \) that was higher than its \( P_f \). Table 2 also summarizes the GFRP bars’ compression contribution at \( P_c \) load level (\( P_{gb} \)), which was determined by multiplying the measured average longitudinal bar strain (\( \varepsilon_{gb} \)) with the total nominal area (\( A_t \)) and elastic modulus (\( E_t \)) of the GFRP bars. GGC-00 yielded the lowest \( P_{gb} \) of 123 kN which translates to a \( P_{gb}/P_c \) of 6.9%. GGC-100 yielded a \( P_{gb}/P_c \) of 7.1% that was marginally higher than GGC-00 while GGC-50 produced a much higher \( P_{gb}/P_c \), equivalent to 8.6%. These results were higher than De Luca et al.’s (2010) findings, with a \( P_{gb}/P_c \) of just 5%, which made them to conclude that the bar contribution could be ignored in evaluating the nominal capacity of the column. This could be related to the lower longitudinal reinforcement ratio (1.0%) and inferior mechanical properties of the GFRP bars they employed in their study.

Table 2 provides the axial deformation at \( P_c \) and \( P_f \), load levels (\( \Delta_c \) and \( \Delta_s \), respectively). The transversely reinforced columns produced \( \Delta_s \) values that were either higher than or equal to that of GGC-00 (7.2 mm), depending on the degree of confinement. GGC-50 had denser arrangement of reinforcement that produced planes of weakness between the cover and core, and hence, this column yielded a larger axial deformation than GGC-100. The \( \Delta_c \) of GGC-50 (19.7 mm) and GGC-100 (11.4) were approximately 2.46, and 1.57 times that of their corresponding \( \Delta_s \), respectively. Obviously, the column with higher volumetric ratio or lower tie spacing demonstrated better deformability performance compared to those with lower volumetric ratios.

<table>
<thead>
<tr>
<th>Column</th>
<th>( P_{gb} ), kN</th>
<th>( P_{fc} ), kN</th>
<th>( P_{gc} ), kN</th>
<th>( \Delta_c ), mm</th>
<th>( \Delta_s ), mm</th>
<th>( \varepsilon_{gc} ), ( \mu \varepsilon )</th>
<th>( \varepsilon_{gc} ), ( \mu \varepsilon )</th>
<th>( \varepsilon_{fg} ), ( \mu \varepsilon )</th>
<th>( \varepsilon_{fg} ), ( \mu \varepsilon )</th>
</tr>
</thead>
<tbody>
<tr>
<td>GGC-8-00</td>
<td>1772</td>
<td>-</td>
<td>123</td>
<td>7.2</td>
<td>-</td>
<td>1424</td>
<td>1647</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>GGC-8-S50</td>
<td>1838</td>
<td>2160</td>
<td>158</td>
<td>8.0</td>
<td>19.7</td>
<td>2183</td>
<td>2116</td>
<td>968</td>
<td>7765</td>
</tr>
<tr>
<td>GGC-8-S100</td>
<td>2063</td>
<td>1717</td>
<td>147</td>
<td>7.2</td>
<td>11.4</td>
<td>1821</td>
<td>1967</td>
<td>730</td>
<td>13131</td>
</tr>
</tbody>
</table>

**Strains in Geopolymer Concrete and GFRP reinforcement**

Figure 7 shows the relationships between the axial load and the average axial strains in the geopolymer concrete, longitudinal GFRP bars, and GFRP spirals. As can be expected, the geopolymer concrete and GFRP bars yielded comparable strains at same load level. Table 2 shows the maximum average strains in the geopolymer concrete at the \( P_c \) load level or the average concrete strain before the cover spalling (\( \varepsilon_{gc} \)). The average \( \varepsilon_g \) (\( \varepsilon_{g-ave} \)) of confined columns was equivalent to 2002 \( \mu \varepsilon \) and was higher than that of GGC-00 (1424 \( \mu \varepsilon \)), owing to the transverse reinforcement that prevented the premature cracking within the specimen and prevented the early buckling of the GFRP bars. Table 2 also summarises the strains in the longitudinal GFRP bars at \( P_c \) load level (\( \varepsilon_{fg} \)). The \( \varepsilon_{fg} \) of GGC-00 was 1647 \( \mu \varepsilon \), which is equivalent to 8.7% of the GFRP bars’ ultimate tensile strain (\( \varepsilon_t \)). GGC-50 and GGC-100 developed \( \varepsilon_t \) of 2116 \( \mu \varepsilon \) and 1967 \( \mu \varepsilon \), respectively, which translate to strain development of 11.2% and 10.4% of \( \varepsilon_t \), respectively. The average \( \varepsilon_{fg} \) (\( \varepsilon_{fg-ave} \)) of the confined columns was 2041 \( \mu \varepsilon \) and was relatively comparable to \( \varepsilon_g-ave \), suggesting the compatibility between the bars and the geopolymer concrete. Hence, equivalency between these materials could be assumed for design and analysis purposes. Furthermore, this average strain value was higher than the design strain limit of 1000 \( \mu \varepsilon \) proposed by Zadeh and Nanni (2013) to avoid exaggerated deflections. In general, marginal transverse strains (\( \varepsilon_{gb} \)) were recorded at lower loads. After exceeding the load equivalent to GGC-00’s \( P_c \), however, relatively higher strains were obtained from GGC-50 (968 \( \mu \varepsilon \)) because of the early spalling of its concrete cover compared to GGC-100 (730 \( \mu \varepsilon \)). At \( P_c \) load level, the results were reversed. The \( \varepsilon_{fc} \) was 13131 \( \mu \varepsilon \) for GGC-8-S100 while GGC-50 produced \( \varepsilon_{fc} \) of 7765 \( \mu \varepsilon \).

**Confinement Efficiency and Ductility Index**

In this study, the column ductility index (\( DI \)) was defined as the ratio of the displacement at 85% of \( P_{peak} \) to the displacement that corresponds to the elastic behavior limit (\( \Delta_{S5}/\Delta_1 \)), as shown in Figure 8. The procedure for determining these displacements was based on Pantelides et al.’s (2013) recommendations. The confinement efficiency (\( CE \)), on the other hand, was computed as the ratio of the compressive strength of the confined column to the compressive strength of the unconfined column (\( f'_{cc}/f'_{cc} \)). The \( f'_{cc} \) was calculated as the peak load divided by the area of the confined geopolymer concrete, which is represented by point C in Figure 9. The \( f'_{cc} \), on the other hand, was equivalent to 0.90\( f'_{cc} \). Table 3 summarises the \( DI \) and \( CE \) values of the tested columns. Based on the experimental results, the ductility index and confinement efficiency increases as the amount of transverse reinforcement increases. Interestingly, these results are consistent with Afifi et al.’s (2013) and Sharma et al.’s (2005) findings for GFRP-RC and conventional RC columns, respectively.
Table 3 Normalised strength, ductility index (DI) and confinement efficiency (CE) of GFRP-RGC and GFRP-RC spiral columns

<table>
<thead>
<tr>
<th>Author</th>
<th>Specimen</th>
<th>$\rho_R$ (%)</th>
<th>DI</th>
<th>CE</th>
<th>$P_{\text{f}} - P_{\text{fg}} / f'_{c} A_g$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Current Study</td>
<td>GGC-50</td>
<td>3.13</td>
<td>2.99</td>
<td>2.13</td>
<td>90.1</td>
</tr>
<tr>
<td></td>
<td>GGC-100</td>
<td>1.57</td>
<td>1.79</td>
<td>1.67</td>
<td>102.7</td>
</tr>
<tr>
<td>Afifi et al. (2013)</td>
<td>G4V-3H80</td>
<td>1.48</td>
<td>1.13</td>
<td>1.37</td>
<td>89.0</td>
</tr>
<tr>
<td></td>
<td>G8V-3H40</td>
<td>2.95</td>
<td>4.75</td>
<td>1.89</td>
<td>89.4</td>
</tr>
<tr>
<td></td>
<td>G8V-3H80</td>
<td>1.48</td>
<td>2.00</td>
<td>1.69</td>
<td>89.2</td>
</tr>
<tr>
<td></td>
<td>G8V-3H120</td>
<td>0.98</td>
<td>1.54</td>
<td>1.32</td>
<td>85.9</td>
</tr>
<tr>
<td></td>
<td>G12V-3H80</td>
<td>1.48</td>
<td>2.45</td>
<td>1.78</td>
<td>89.4</td>
</tr>
<tr>
<td>Pantelides et al. (2013)</td>
<td>13GLCTL</td>
<td>1.91</td>
<td>1.70</td>
<td>1.76</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>14GLCTL</td>
<td>1.91</td>
<td>3.60</td>
<td>1.59</td>
<td>-</td>
</tr>
</tbody>
</table>

CONCLUSIONS

This study investigated the behavior of geopolymer concrete spiral columns reinforced longitudinally and transversely with GFRP bars. From the experimental results, the following conclusions were drawn:
The compression contribution of the GFRP bars (with a reinforcement ratio of 2.43%) with respect to column capacity varied from 6.9% to 8.6%, with an average value of 7.5%.

The column with a smaller spiral pitch (50 mm on-center) yielded higher load carrying capacity and failed in a more ductile manner compared to the column with a larger spiral pitch (100 mm on-center). In addition, the former column yielded ductility index and confinement efficiency that were 67% and 28%, respectively, higher than that of the latter column.

The GFRP-RGC circular columns yielded a relatively higher normalized strength (96.4%) compared to GFRP-RC circular columns (88.6%). This could be attributed to the higher elastic modulus of geopolymer concrete (33 GPa) compared to normal concrete (29 GPa) of the same grade (38 MPa), resulting in a better compatibility in the GFRP-RGC system than in the GFRP-RC system. Further studies, however, are needed to validate this generalization.

It can be inferred, therefore, that the GFRP-RGC system could be adopted for the fabrication of compression members, particularly when corrosion resistance, electromagnetic transparency, material greenness, durability, and sustainability are sought.

ACKNOWLEDGMENTS

The authors would like to express their special thanks to V-ROD® Australia for providing the materials as well as to the Natural Science and Engineering Research Council of Canada (NSERC) and the technical staff of the structural lab at University of Southern Queensland.

REFERENCES

ABSTRACT

The experimental investigations conducted to study the behaviour of ultra-high performance steel fibre concrete (UHPSFC) reinforced with fibre reinforced polymer (FRP) material is presented in this paper. The investigations include the compressive behaviour of UHPSFC confined with FRP tubes and flexural behaviour of UHPSFC beams reinforced with internal FRP bars. 6 cylindrical specimens of UHPSFC confined with Carbon FRP (CFRP) and Basalt FRP (BFRP) tubes were tested under compression. Two full scale UHPSFC beams of 3250mm span reinforced with CFRP and Glass FRP (GFRP) bars were tested under four point bending. The compression behaviour of UHPSFC is significantly improved by the confinement provided by FRP material in terms of both strength and ductility. The compressive strength of UHPSFC confined with FRP tubes is 1.35 times greater the unconfined UHPSFC cylinders. The ultimate failure strain of FRP confined UHPSFC cylinders is found to be 3 to 4.5 times greater than the unconfined ones. The beams reinforced with FRP bars found to have greater load capacity than the steel bar reinforced beams with same percentage of reinforcement. The experimental results reflect the potential of FRP composites to build hybrid structural members with UHPSFC material.

KEYWORDS

Ultra-high performance concrete, FRP, confinement, UHPC beams.

INTRODUCTION

Ultra-high performance steel fibre concrete (UHPSFC) is an advanced cement composite material, which is characterised by high strength, ductility, durability and fracture toughness. The higher strength characteristics of the material enables to design innovative structures with significantly smaller cross-sections (Graybeal 2005) and without conventional steel bar reinforcement (Voo et al. 2012). In last decade, several studies investigated the material properties (Graybeal 2007, Hassan et al. 2012) and structural applications of UHPSFC (Yang et al. 2010, Fisher et al. 2002). Very few studies investigated the compressive behaviour of confined UHPSFC. The improvement provided by the confinement of normal strength concrete in terms of strength and ductility is well understood. It is also well known that due to the low dilation capacity of high strength concrete the confinement leads to insignificant improvement in its strength and ductility (Mandal et al. 2005). The UHPSFC has a considerable dilation capacity which is provided by the internal steel fibres, considering which Zohrevand and Miriran 2011 investigated the compressive behaviour of FRP confined UHPSFC. It was found that FRP confinement lead to significant improvement in strength and strain capacities of UHPSFC. It was reported that the available stress strain models developed for FRP confined normal strength concrete are not reliable to predict the stress strain behaviour of FRP confined UHPSFC. It was recommended that further research is required to investigate the behaviour of FRP confined UHPSFC which includes the practical confinement ratios to develop reliable FRP confined stress strain models of UHPSFC.

In past few years, the new type of reinforcing materials in the form of fibre reinforced polymer (FRP) bars emerged as an effective way to enhance the corrosion resistant of structures. The replacement of steel bars with FRP material is found to have several advantages such as high corrosion resistance, high strength to weight ratio, non-conductivity and ease of handling (Ehsani et al. 1997). Due to their high strength, the use of FRP bars with UHPSFC can provide a new way of optimizing the structural members. The objective of this study is to investigate the behaviour of hybrid members made up of two advanced materials of FRP and UHPSFC subjected to flexural and compressive loadings. To study the flexural behaviour, the composite beams of UHPSFC reinforced with
different materials such as glass FRP bars, carbon FRP bars, and conventional steel bars were tested. The load displacement behaviour, cracking pattern and failure modes of the beams with different rebar material is compared. To investigate the compressive behaviour of FRP confined UHPSFC, 6 cylindrical specimens of UHPSFC confined with Carbon FRP (CFRP) and Basalt FRP (BFRP) tubes were tested.

EXPERIMENTAL PROGRAM

Test Specimens and materials

To investigate the compressive behaviour of FRP confined UHPSFC, a total of 6 cylinders of 100mm (diameter) x 200mm (height) were tested. Additionally 3 unconfined cylinders of identical dimensions were also tested. The tubes were made by using manual wet layup procedure and cured for 7 days before the concrete casting. 4 circumferential layers of FRP material was provided to the confined specimens either in the form of carbon FRP or basalt FRP. The properties of the CFRP and BFRP fabric used are shown in Table 1, which is extracted from the coupon tests. The specimens are designated according to the FRP material used for confinement. The unconfined specimens are denoted with a letter U followed by the number, which denotes the identical specimen number such as U-1 is unconfined cylinder specimen number 1. Similarly confined specimens are designated as CC-1 or CB-1 as confined cylinder with CFRP or BFRP material, respectively.

<table>
<thead>
<tr>
<th>Material</th>
<th>Ultimate strength (MPa)</th>
<th>Ultimate strain (%)</th>
<th>Elastic Modulus (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP-Fabric</td>
<td>1820</td>
<td>1.03</td>
<td>177</td>
</tr>
<tr>
<td>BFRP-Fabric</td>
<td>1204</td>
<td>1.97</td>
<td>63</td>
</tr>
<tr>
<td>CFRP-Rebar</td>
<td>1800*</td>
<td>1.32*</td>
<td>144*</td>
</tr>
<tr>
<td>GFRP-Rebar</td>
<td>1100*</td>
<td>1.73*</td>
<td>62*</td>
</tr>
<tr>
<td>Steel-Rebar</td>
<td>625</td>
<td>17.0</td>
<td>195</td>
</tr>
</tbody>
</table>

* reported by manufacturer

To investigate the flexural behaviour of UHPSFC beams reinforced with FRP rebar material, 2 full-scale beams were tested. Additionally 1 full scale beam as a control specimen with identical cross section and span with steel rebar as reinforcing material was also tested for comparison purposes. The details of all the test beams are given in Table 2. All the beams were of 250 x 250mm cross-section with a span of 3250mm. The beams were designated with a first letter as “UHB” followed by a letter representing the name of the rebar material used and then followed a number representing the diameter of rebar used as tensile reinforcement. The letter “ST”, “GF”, and “CF” represents the reinforcing bar material as steel bar, GFRP bar, and CFRP bar. For example the beam with steel rebar of 20mm diameter is designated as UHB-ST-20. To assure the flexure failure mode of the beam, the shear reinforcement was also provided which consisted of 8mm diameter stirrups spaced at 90mm centre to centre up to the load points as shown in figure 1. The properties of the reinforcing bar materials as reported by manufacturer is shown in Table 1.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Dimension B x D (mm)</th>
<th>Reinforcing material</th>
<th>Bar diameter (mm)</th>
<th>Number of bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>UHB-ST-20</td>
<td>250 x 250</td>
<td>Steel</td>
<td>20</td>
<td>3</td>
</tr>
<tr>
<td>UHB-GF-20</td>
<td>250 x 250</td>
<td>GFRP</td>
<td>20</td>
<td>3</td>
</tr>
<tr>
<td>UHB-CF-12</td>
<td>250 x 250</td>
<td>CFRP</td>
<td>12</td>
<td>3</td>
</tr>
</tbody>
</table>

UHPSFC Mix

In this study, two type of UHPSFC mix were used. The mix were developed at The University of Adelaide using locally available raw materials. The UHPSFC mix-1 consists of specially graded sands so as to optimize the packing density of the mix. The UHPSFC mix-2 utilises only materials commonly used in the manufacture of
conventional normal strength concrete with the purpose of minimising material costs, the details of which can be found in (Sobuz et al. 2016). The detail proportioning of materials for the UHP SFC mix adopted in this study are given in Table 3. The binder consisting of sulphate resistant cement and silica fume was used in both the mix. The sands used in mix-1 were in the form of silica powder with an average size of 50μm and silica sand grade 30/60 with an average size of 400μm. For mix-2, river washed sand from local quarry was used as filler and no coarse aggregates were used in both the mix. The third generation liquid based High range water reducing admixture (HRWR) of Sika Viscocrete was used to achieve the workability. The steel fibres used were hooked end steel fibres of 35mm in length with aspect ratio of 64 and yield strength of 1100MPa reported by the manufacturer. The amount steel fibre added was 2.25% by volume.

All the dry constituents were electronically weighed and mixed in dry state in horizontal pan mixer for at least 3 minutes. The water and HRWR were mixed together and added into the material and allowed to mix until the materials turns into a flow-able consistent paste. The steel fibres are then added and mixed for further 5 minutes.

Table 3 Mix proportions

<table>
<thead>
<tr>
<th>Mix</th>
<th>Cement</th>
<th>Silica fume</th>
<th>Quartz powder</th>
<th>Silica Sand</th>
<th>River sand</th>
<th>HRWR</th>
<th>Water</th>
<th>Steel fibre (by Vol.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.0</td>
<td>0.266</td>
<td>0.5</td>
<td>0.5</td>
<td>1.0</td>
<td>0.045</td>
<td>0.15</td>
<td>2.25%</td>
</tr>
<tr>
<td>2</td>
<td>1.0</td>
<td>0.266</td>
<td>-</td>
<td>-</td>
<td>0.0565</td>
<td>0.17</td>
<td>2.25%</td>
<td></td>
</tr>
</tbody>
</table>

Fabrication

The FRP tubes were manufactured using manual wet layup process by wrapping the epoxy resin impregnated sheets around the cylindrical shape Styrofoam which had a diameter of 101mm and 220mm height. The fabrics of width 205mm were cut to a total length of 4 layers with an additional overlap length of 100mm. The epoxy resin is applied to whole length and on both sides of the fabric with the help of a roller brush and wrapped to the Styrofoam tube in the hoop direction. After 24 hours the hardened FRP tubes were removed from Styrofoam cylinders and cured for 7 days at room temperature. The UHPSFC was filled into the tubes and after 24 hours the specimens were placed and cured in a fog room for at least 56 days until testing.

The beams are fabricated one at a time due to the large amount of material required. The horizontal mixer of 1 ton capacity installed at The University of Adelaide is used for the onsite manufacturing of the structural members. The wooden formwork is built for the casting of the beams and all the beams are casted horizontally. The specimens required for the material tests are also casted for each batch of concrete. After 2 hours of casting all the specimens are covered with wet hessian and plastic sheets for at least 24hours. The formwork is then removed and the beams are cured with wet hessian covered with plastic sheets for at least 28 days before testing.

Instrumentation and Testing procedure

Compression test

All the confined and unconfined cylindrical specimens were provided with 2 electrical strain gauges of 30mm length to record the axial strains. Four equally spaced 5mm strain gauges were also attached to the confined specimens to measure the lateral/hoop strains in the FRP material. Additionally 4 linear variable differential transformers (LVDT) were used to measure the platen-to-platen displacement so as to measure the strains of the whole specimen. The uniaxial compressive load was applied at a constant rate of 0.05mm/minute throughout the test for which a universal testing machine of 5000kN capacity was used. The load, strain and displacement data were recorded continuously at the rate of two readings per second.

Beam Test

The beams were subjected to two equal concentrated loads applied symmetrically at a distance of 250mm from the mid span as shown in figure 1, so as to induce pure bending stresses in the beam between the load points. The load was applied through a single hydraulically actuated jack to the steel section placed on the top face of the beam with its supports 500mm apart. The beams were tested until collapse with monotonically applied load and the corresponding deflections and strains were recorded simultaneously.
TEST RESULTS AND DISCUSSION

Compressive behaviour

Table 4 presents the summary of tests results in terms of ultimate strength and strains of unconfined and confined specimens. The unconfined compressive stress strain curve obtained from experimental tests for the UHPSFC is shown in figure 2a. The average compressive strength of the UHPSFC is 140MPa and the corresponding axial strain is 0.4%. It can be seen (figure 2a) that the stress strain relation is almost linear up to the peak load followed by a sudden drop of load until a stable stress plateau is attained. Figure 2b shows the compressive stress strain curve of CFRP confined cylindrical specimens along with the unconfined ones. The CFRP confined specimens follows the same trend of linear stress strain up to the attainment of peak strength as is found in unconfined cylinders. The peak strength is increased by 36% by the CFRP confinement when compared to the unconfined compressive strength. A drop in strength after attaining the initial peak strength can be seen in the stress strain plots (Figure 2b), which is due to the softening of the inner core. The behaviour is similar to the high strength concrete (HSC) reported by (Xie and Ozbakkaloglu 2015, Mandal et al. 2005) which is caused by the brittle nature of HSC and ultra HSC. The softening leads to 8% decrease in the strength which is followed by the hardening branch of stress strain curve. The stress plateau of the hardening branch attains almost similar strength as that of initial peak strength followed by similar trend of softening and hardening until the rupture of CFRP tube.

The ascending branch of stress strain curve of BFRP confined specimen (Figure 2c) remains linear typically up to 85% of the unconfined strength after which it follows curvilinear path to attain the peak strength followed by the softening branch. The BFRP confined specimen shows 16% gain in strength when compared with the unconfined ones. At softening the strength drop is almost equal to 16% of the peak strength which means that the softening occurs till the strength is reduced to the unconfined strength of the material. After softening the material is unable to recover its peak strength which is attributed due to the less confinement effectiveness of BFRP because of its low elastic modulus compared to CFRP. The strength increase provided by the BFRP confinement is not significant in comparison to the CFRP confinement. In terms of ductility, BFRP confinement showed superior performance than CFRP. The results show that the compressive stress strain behaviour of CFRP confined UHPSFC is very similar to the elasto-plastic behaviour of steel. Such a behaviour is extremely desirable for the design of compression members located in the regions of high seismicity.

Flexural behaviour

The deflection is measured at mid span for all the beams. Figure 3 shows the load versus mid span displacement relationship obtained from the tests. The load-displacement curves indicate that the relation can be divided into three to four distinct regions, namely initial linear zone before first cracking, yield load at the yielding of rebar, peak load and collapse load of beam. The first stage of load displacement curve corresponds to the high bending stiffness of the uncracked section response of the beams. The first cracking load for beam UHB-ST-20 is 20kN and for UH-BG-20, UH-BC-12 is 26kN. After the first cracking load, the reduction in bending stiffness is observed due to the initiation of micro cracks. The load displacement curve observed to be linear with reduced stiffness until the yield load is attained. The reduction in stiffness is found to be dependent on the stiffness provided by the reinforcing bar material that is a factor of elastic modulus of bars and percentage of reinforcement provided.
Table-4 Summary of unconfined and confined compression test results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Ultimate strength (f_{cu}) (MPa)</th>
<th>Average Ultimate strength (MPa)</th>
<th>Ultimate axial strain</th>
<th>Ultimate hoop strain</th>
<th>Ratio of Confined to Unconfined strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>U-1</td>
<td>140</td>
<td></td>
<td>0.39</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>U-2</td>
<td>145</td>
<td>140</td>
<td>0.40</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>U-3</td>
<td>138</td>
<td></td>
<td>0.39</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>CC-1</td>
<td>190</td>
<td>196</td>
<td>1.24</td>
<td>0.69</td>
<td>3.7</td>
</tr>
<tr>
<td>CC-2</td>
<td>196</td>
<td>192</td>
<td>0.98</td>
<td>0.86</td>
<td>1.37</td>
</tr>
<tr>
<td>CC-3</td>
<td>190</td>
<td></td>
<td>1.10</td>
<td>0.85</td>
<td>1.37</td>
</tr>
<tr>
<td>CB-1</td>
<td>154</td>
<td></td>
<td>1.70</td>
<td>1.89</td>
<td>1.16</td>
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<tr>
<td>CB-2</td>
<td>161</td>
<td>163</td>
<td>1.78</td>
<td>1.83</td>
<td>1.16</td>
</tr>
<tr>
<td>CB-3</td>
<td>175</td>
<td></td>
<td>1.55</td>
<td>1.90</td>
<td>1.16</td>
</tr>
</tbody>
</table>

The failure of beams UHB-ST-20 and UHB-CF-12 initiated due to the yielding of the reinforcing bars, whereas beam UHB-GF-20 yielded due to the crushing of concrete. The beams with steel bars is found to have long ductile plateau after the yield load is attained with the final collapse is due to the rupture of the reinforcing bars. The beam UHB-CF-12 with CFRP rebar observed to have negligible ductility after the yield load and the beam failed in a sudden manner due to rupture of CFRP bars. The failure of beam UHB-GF-20 initiated with the crushing of concrete. Although the beam has less ductility, the load drop is gradual as the crushing of UHPSFC is progressed which is different to the sudden and brittle failure observed for cylindrical specimens under pure compression.

Table 5 summarises the experimental results obtained from the beam tests. The results indicate that only 1.5% of reinforcement is required for high modulus GFRP material to obtain the compression failure mode of the beams whereas the beam with same percentage of steel bar reinforcement showed tension failure mode due to the rupture of rebar. The beam UHB-CF-12 with only 0.5% of reinforcement attained the similar stiffness as of beam UHB-GF-20 with 1.5% of reinforcement. The load carrying capacity of UHB-CF-12 is found to be 18% greater than UHB-ST-20 which has 3 times reinforcement percentage when compared to UHB-CF-12.

CONCLUSIONS

The behaviour of UHPSFC reinforced with FRP materials in the form of tubes and reinforcing bars subjected to compressive and flexural loadings is investigated in this study. The compression behaviour is investigated by testing 6 FRP confined UHPSFC cylinders and flexural behaviour is investigated by testing 3 large scale beams. The following conclusions are drawn from the experimental investigations:

The compressive strength of UHPSFC was increased by 36% and 16% when confined with CFRP and BFRP material, respectively. The FRP confinement significantly improved the ductility of UHPSFC where the ultimate axial strains were increased by a factor of 3 for CFRP confinement and by a factor of 4.5 for BFRP confinement.
The UHPSFC beams reinforced with conventional steel rebars exhibit very ductile failure. The fibres resisted the opening of the cracks and contributed toward the increased load carrying capacity of beams even after the steel bars are yielded. The beams reinforced with FRP materials had reduced stiffness when compared with beams with steel rebars. However, the cracking pattern was observed to be similar for both type of reinforcing materials. Due to the high strength of FRP materials it was found that only 1.5% of reinforcement is required by GFRP rebars to obtain the compression failure mode of the beam, whereas the beam with same percentage of steel bar reinforcement showed tension failure mode due to rupture of rebar.

The beam with 0.5% of CFRP reinforcement had tension mode of failure whereas the stiffness achieved is identical to the beam with 1.5% of GFRP reinforcement and 18% greater load capacity than the beam with 1.5% of steel reinforcement. Further tests are required to obtain the percentage of CFRP reinforcement required to change the mode of failure from tension to compression. The experimental results reflects the potential of FRP composites to build hybrid structural members with UHPSFC material.

ACKNOWLEDGEMENTS

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REFERENCES

EXPERIMENTAL AND NUMERICAL STUDY ON THE SHEAR BEHAVIOUR OF
GEOMETRICALLY SIMILAR FRP RC BEAMS

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Emanuele Zappa

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ABSTRACT

This paper discusses an experimental study examining size effect in GFRP RC beams with overall height varying from 260 mm to 460 mm, but otherwise similar in geometry and material properties. All beams were unreinforced in shear and designed to ensure shear failure. 3D DIC was employed alongside strain and displacement transducers to gain additional insight into the development and degradation of shear resisting mechanisms. The experimental measurements will be used to calibrate a numerical model and validate its reliability and performance. The results of this study can assist in improving existing shear design recommendations for FRP RC beams.

KEYWORDS

FRP, RC beams, shear, size effect, experimental study, numerical analysis.

INTRODUCTION

The shear performance of reinforced concrete (RC) members is governed by the development of a complex set of mechanisms. Shear stresses can, in fact, be transferred through a section of an element by the portion of concrete in compression, aggregate interlock, friction along a diagonal shear crack, dowel action of the longitudinal reinforcement and, when provided, the transverse shear reinforcement. Various studies have examined the shear behaviour of steel RC beams and have shown that member size directly affects the magnitude of the shear resisting mechanisms. In particular, Kani (1967) examined the performance of a series of beams by varying the depth of their cross section, \( d \), while keeping the steel reinforcement ratio constant. The results of this study (Figure 1a) show that the normalised shear strength of the tested beams decreased with the increase of \( d \). Such a trend, however, seemed to be less pronounced in slender beams. Various researchers (Shioya et al. 1990, Collins and Kuchma 1999, Frosch 2000 and Lubell et al. 2004) proved that size effect may be attributed mainly to a reduction in the resistance offered by aggregate interlock as a result of the larger cracks that develop in larger elements (Figure 1b). It was also found that crack widths increase linearly with crack spacing and that there is a direct relationship between crack spacing and depth of the beam (Shioya et al. 1989).

Over the past decades a great deal of attention has been put on fibre reinforced polymer (FRP) bars, which moved rapidly from being a novel alternative to steel reinforcement to a well-established practice. FRP bars are characterized by a linear elastic mechanical response and, consequently, the structural behaviour of FRP RC is different than that of steel RC. Although shear in FRP RC beams is transferred through the development of the same resisting mechanisms as for steel RC, their magnitude and individual contributions to overall shear capacity needs to be carefully reassessed. In particular, under similar loading conditions FRP RC elements can develop much higher deformations, thus exhibiting wider cracks than their equivalent steel RC counterparts. In turn, these larger deformations result in a reduced portion of concrete in compression resisting shear and in a weakened aggregate interlock along the cracks. In addition, due to the low transverse shear resistance of the FRP bars, the contribution of dowel action is negligible (Razaqpur et al. 2004).

While the overall shear behaviour of FRP RC is well documented (e.g., Guadagnini et al. 2006, Yang et al. 2015), research on the effect of element size on shear strength is still limited (Alam and Hussein 2012, Ashour and Kara 2014). This paper discusses the results of the first stage of a larger testing programme that aims to examine size effects in FRP RC beams.
EXPERIMENTAL PROGRAMME

Three geometrically similar beams were tested as part of an experimental programme designed to investigate the size effect in shear FRP RC beams without shear reinforcement. The longitudinal reinforcement ratio, $\rho_{lf}$, and shear span to depth ratio, $a/d$, were similar for all specimens (Table 1), while the overall depth, $d$, varied from 260 mm to 460 mm (Figure 2). The longitudinal reinforcement consisted of 12.7 mm GFRP bars in tension, while 6 mm basalt FRP (BFRP) bars were used as compression and skin reinforcement. The overall length of the all beams was 2500 mm. All of the specimens were cast using the same batch of ready mix concrete and the average concrete strength, $f_{cy}$, for each beam is also reported in Table 1 as measured on the day of testing.

Two tests were performed on each of the specimens. During the first test (Phase I, tests GB54 and GB58) the position of the load was selected so as to induce shear failure along the shorter of the shear spans, $A$ in Figure 2 (top), while maintaining relatively low average shear stress levels along the adjoining shear span ($B+C$) to ensure that this would remain relatively undamaged. Before the re-tests (Phase II, tests GB55 and GB59), the damaged portion of each beam (shear span $A$) was cut, and post-tensioned metal straps (PTMS) were wrapped along span $C$ to provide the required additional shear strength and avoid failure in this zone (Figure 2, bottom). Given that the two shear spans of specimens GB56 and GB57 were similar, PTMS were used in each phase of testing to strengthen one of the shear spans and ensure failure of the instrumented shear span.

Material properties

The summary of geometric and material properties of the test beams is presented in Table 1, where $a$ is the shear span, $f_{cy}$ is the concrete cylinder compressive strength (taken as 80% of the compressive strength obtained from testing 100 mm cubes), $E_f$ is the GFRP modulus of elasticity, $\rho_{lf}$ is the longitudinal reinforcement ratio, $F_{ult}$ is the ultimate applied load, $V_{exp}$ is the corresponding shear load at failure, $V_{norm}$ is the shear strength at failure normalized by the concrete compressive strength, and $\delta_{max}$ is the maximum deflection under the loading point.

<table>
<thead>
<tr>
<th>Beam</th>
<th>$a$ [mm]</th>
<th>$a/d$</th>
<th>$f_{cy}$ [MPa]</th>
<th>$E_f$ [GPa]</th>
<th>$\rho_{lf}$ [%]</th>
<th>$F_{ult}$ [kN]</th>
<th>$V_{exp}$ [kN]</th>
<th>$V_{norm}$ [MPa]</th>
<th>$\delta_{max}$ [mm]</th>
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</thead>
<tbody>
<tr>
<td>GB54</td>
<td>900</td>
<td>2.70</td>
<td>30.16</td>
<td>46</td>
<td>0.76</td>
<td>51.46</td>
<td>31.32</td>
<td>0.11</td>
<td>12.0</td>
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<tr>
<td>GB55</td>
<td>900</td>
<td>2.70</td>
<td>30.16</td>
<td>46</td>
<td>0.76</td>
<td>132.54</td>
<td>35.48</td>
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<td>GB56</td>
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<td>2.59</td>
<td>38.04</td>
<td></td>
<td>0.78</td>
<td>85.57</td>
<td>43.90</td>
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</tr>
<tr>
<td>GB57</td>
<td>1120</td>
<td>2.59</td>
<td>36.64</td>
<td></td>
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<td>99.35</td>
<td>49.96</td>
<td>0.13</td>
<td>10.1</td>
</tr>
<tr>
<td>GB58</td>
<td>620</td>
<td>2.66</td>
<td>36.64</td>
<td></td>
<td>0.73</td>
<td>51.02</td>
<td>37.26</td>
<td>0.18</td>
<td>20.5</td>
</tr>
<tr>
<td>GB59</td>
<td>620</td>
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<td></td>
<td>0.73</td>
<td>91.23</td>
<td>57.56</td>
<td>0.27</td>
<td>26.0</td>
</tr>
</tbody>
</table>

Instrumentation

The applied load was measured using the internal force transducer in the actuator, while displacement and strains were recorded using LVDTs and strain gauges, respectively. The displacement was measured under the load point, where the maximum deflection was expected, and at the supports to determine the net deflection of each specimen.
Strain gauges were placed along the longitudinal reinforcement on the test side of the beam, which was also monitored with Digital Image Correlation (DIC). All details of the instrumentation are presented in Figure 2.

![Figure 2 a) Reinforcement scheme; b) load setup and instrumentation arrangement of the FRP RC beams without shear reinforcement (dimensions in mm)](image)

**DIC setup**

DIC was employed to obtain the full field strain deformations within the shear span of the beams, and in particular, a three dimensional (3D) configuration was used to eliminate the effects of possible out of plane displacement of the specimens during testing (i.e., generation of apparent strains) (Di Benedetti et al. 2015). Images were acquired with two CMOS (complementary metal-oxide semiconductor) digital cameras having a 4272×2848 pixel resolution (Canon EOS 1100D) and equipped with zoom lenses with F-number and focal length of 3.5-5.6 and 18-55 mm, respectively (Canon EF S 18 55mm f/3.5 5.6 IS II). The two cameras were rigidly connected and positioned 2 m apart from the specimen and a light-emitting diode (LED) lamp was used to uniformly illuminate the measurement surface. The stereo vision system was calibrated by taking images of a known pattern with different positions and orientations. During the load test, the shutter was remotely triggered three times every 10 seconds by the data acquisition system recording the point wise sensors in order to synchronize all data. The field of vision (FOV) was selected to accommodate the whole test shear span (Figure 2, A and B in Phase I and II, respectively), extending below the specimens to account for the expected maximum deflection at failure. After whitewashing the beams, speckles with an approximate diameter of 1 mm were spray painted in the region of interest.

**EXPERIMENTAL RESULTS**

**Moment capacity and shear strength**

The summary of the test results is listed in Table 1. In general, an increase in the ultimate moment capacity was observed with an increase in the overall depth of the members, \(d\) (solid lines in Figure 3a). The second test on each beam (dashed lines) resulted in overall higher capacity, which can be attributed to the fact that shortening the clear span of the beams to remove the damaged portion of the specimen after the first test (GB54 and GB58), partially changed the internal resisting mechanisms affecting both stiffness and ultimate behaviour. For example, in the test of GB55, the short span (\(C\) in Figure 2) contributed to the overall shear capacity by engaging a strut-and-tie action, resulting in a much higher shear resistance than that provided by GB54 (first test).

The experimental shear strength \(V_{exp}/bd\) was normalized by the concrete strength, \(\sqrt{f_{ck}}\), and was plotted against the members’ effective depth, \(d\) (Figure 3b). The results show that the normalized shear strength decreases with an increase in \(d\), which is in good agreement with available literature (Ashour and Kara 2014). In particular, a drop in shear strength of about 60% was observed between tests GB59 and GB56, when overall depth increases from 260 mm to 460 mm, respectively. This could be explained by the opening of wider diagonal shear cracks causing a reduction of shear resisting mechanism provided by mechanical interlocks.
In order to gain a more in-depth understanding of the shear cracking mechanism, and how it affects shear resistance, the DIC measurements taken along the test shear span of beams GB54 and GB56 are shown in Figure 4a and 4b, respectively. In both specimens, the critical diagonal cracks developed from the tip of the vertical flexural crack closer to the support (extensometers 3 and 2 in Figure 4a and 4b, respectively), and propagated towards the loading point at increasing load levels.

The width of the shear crack was measured at eight points perpendicularly to the crack direction (ext. 1-8 marked on the counter map inserts showing horizontal strains) and plotted against the percentage of the ultimate load of each respective beam. Flexural cracking at the monitored location started at a load of about 40% of ultimate, as evidenced by the increase in crack width captured by the DIC system. With a further increase in the applied load, the flexural crack turned gradually into an inclined crack and progressed towards the point of application of the external load. Just before failure occurred (at 87% and 93% of the ultimate load of beams GB54 and GB56, respectively), the inclined crack extended backward below the tip of the original flexural crack resulting into a significant increase in maximum crack opening (ext. 1-5). The results show that the largest crack opening at failure occurred above the tip of the flexural crack and was equal to 1.2mm (ext. 2) for beam GB56, while a maximum crack width of 1mm was observed in beam GB54 (ext.3).

**NUMERICAL INVESTIGATION**

The tests on GB54, GB56 and GB58 were numerically simulated using FEMIX, a finite element program that implements a 3D multidirectional fixed smeared crack model (Gouveia 2011). The specimens were discretized using 8 node plane-stress finite elements with a Gauss-Legendre (GL) integration scheme of $2\times2$ integration points (IP). An energy tolerance of $1\times10^{-3}$ was used as convergence criteria and a maximum of 2 sets of cracks in each
IP was allowed to form. The reinforcement was simulated with perfectly bonded, 2-node 2D embedded cable elements and a GL integration scheme of 2 IP. All analyses were executed with a modified Newton-Raphson iterative algorithm where the stiffness matrix was updated in the first iteration of each load increment. The load was applied in displacement control, with the arc-length technique, up to the ultimate beam’s deflection, at which stage it was no longer possible to ensure convergence. Poisson’s ratio ($\nu = 0.20$), initial Young’s modulus ($E_c = 24830$ MPa), parameters of the trilinear softening curve ($\xi_1 = 0.006$, $\alpha_1 = 0.5$, $\xi_2 = 0.1$, $\alpha_2 = 0.05$, $G_{f,1} = 0.05$ N/mm, Figure 5a), parameters of the shear softening curve ($\beta = 0.5$, $\tau_{cr} = 0.5$ MPa, $G_{f,s} = 0.025$ N/mm, Figure 5b), and the threshold angle ($\theta_{th} = 30^\circ$) were maintained constant for all analyses. Conversely, the concrete compressive and tensile strength varied for the three beams to reflect the values obtained from the experimental characterization. The modulus of elasticity of the reinforcement was 46 GPa.

Figure 5 Concrete softening diagram for fracture mode: (a) I and (b) II

Figure 6 shows a comparison between experimental and numerical analyses. In particular, Figures 6a-c show the deflection calculated under the loading point as a function of the applied load, while Figures 6d-f compare the crack patterns observed experimentally at failure with those obtained numerically (only fully open cracks are represented, i.e. their energy exceeded the specific fracture energy).

Figure 6 Comparison of experimental and numerical results: Load – deflection curves for beams (a) GB58 (b) GB54 (c) GB56, Crack pattern (d) GB58 (e) GB54 (f) GB56

The experimental shear failure was well captured by the numerical simulations while the stiffness of the overall load-deflection response was generally over-estimated. The higher predicted initial stiffness can be attributed to the fact that a certain degree of initial cracking was already induce in the specimens during the setup operations.
In the elasto-cracked stage, the stiffness of specimen GB58 (smallest effective depth) was well captured, while the stiffness of specimens GB54 and GB56 deviated from the experimental results. In particular, the higher the effective depth, the higher the deviation between numerical and experimental stiffness, suggesting that this phenomenon was not being well captured by the constitutive model. To account for this effect, the parameters defining the crack shear softening diagram, mainly fracture mode II, should directly consider the crack opening.

CONCLUSIONS

The results from a first phase of an experimental programme on size effect of shear critical FRP RC beams have been reported in this paper. A 3D-DIC was employed to enable a more accurate assessment of the local cracking and deformation behaviour throughout the whole load history. A FEM numerical model was developed to try and capture the behaviour of the tested beams as well as the crack development.

The experimental tests confirmed that beam depth plays an important role in determining the shear behaviour of FRP RC beams, which can already be significant for a change in overall depth from 260 mm to 360 mm (typical of most laboratory tests).

Although only preliminary results were presented for the analysis of crack development using DIC, this promising measuring technique can enable a better understanding of local deformation phenomena and could be used to obtain critical damage parameters to inform the development of more reliable numerical models, which still to date cannot simulate the complex behaviour of concrete elements subjected to brittle failure modes.

REFERENCES


Frosch, R.J. (2000). “Behavior of Large-Scale Reinforced Concrete Beams with Minimum Shear Reinforcement”, *ACI Structural Journal*, 97(6), 814-820.


AN EXPERIMENTAL STUDY ABOUT THE SHEAR BEHAVIOR OF HIGH STRENGTH CONCRETE BEAMS REINFORCED WITH FRP BARS

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ABSTRACT

Recently, much research efforts has been directed towards investigating the shear behavior of concrete beams reinforced with fibre reinforced polymer (FRP) bars. However, there is still a demand for more studies to solve this issue, especially in the case of high strength concrete (HSC) members. This paper presents an experimental study about the shear behavior of HSC beams longitudinally reinforced with carbon fibre reinforced polymers (CFRP) bars. A total of thirteen simply supported HSC beams with or without steel stirrups were fabricated and tested under four-point monotonic loading until failure. A rectangular cross section of width (b=120 mm) and effective depth (d=224 mm) is adopted for all beams. The main investigated parameters include the shear span-to-depth ratio (a/d), the longitudinal reinforcement ratio (ρf%), the transverse reinforcement ratio (ρv%), and the longitudinal reinforcement type (conventional steel bars or CFRP bars). All beams were designed so that failure would occur due to shear. The mode of failure, the cracking load, the ultimate shear strength, and the midspan deflection are presented. It was found that the shear failure of HSC beams reinforced with CFRP bars occurs without warning; this may be attributed to the low elasticity modulus of CFRP bars. Addition of longitudinal steel bars to the flexural reinforcement of the tested beams improved both their ultimate shear capacity and their ductility. The ultimate shear capacity is basically dependent on a/d and ρv%.

KEYWORDS

CFRP bars, high strength concrete, shear, experimental, monotonic loading.

INTRODUCTION

The use of fibre reinforced polymer (FRP) bars in reinforced concrete (RC) members has become extremely popular in the last decades as they provide anticorrosive alternative for steel reinforcement in addition to their high strength-to-weight ratio. FRP bars have superior tensile strength compared to steel; they behave in a linearly elastic manner up to failure (ACI 440.1R-06). This makes the concrete reinforced with FRP bars more susceptible to brittle failure especially when used in high strength concrete (HSC) members. The use of HSC is rapidly increasing in buildings, bridges, and other structures due to its superior strength and stiffness. The use of HSC in combination with FRP reinforcement allows a better use of high stress and strain properties of FRP bars. However, an increase in the strength of the concrete produces an increase in its brittleness and smoother shear failure surfaces leading to a reduction in the contribution of aggregate interlock and hence a reduction in the shear force carried by concrete. These combined effects can ultimately lead to brittle shear failures. Several studies on the shear capacity of FRP reinforced members without shear reinforcement have indicated that the shear strength is influenced by the stiffness of the tensile reinforcement, the shear span to depth ratio, the tensile strength of concrete and the depth of the member (Sonobe, et al., 1997; Michaluk, et al., 1998; Tureyen; Frosch, 2003; Gross, S., Yost, et al., 2003; Ashour, A., 2006; Alam, M. S., 2010; El-Sayed, 2006b and Razaqpur, et al., 2011). The behavior of FRP bars is brittle with no yielding at failure. Since the shear capacity prediction is essential in the design of FRP reinforced concrete members, the ACI 440.1R-06 guide recommends that such members be designed as over reinforced sections and recognizes that the shear behavior of FRP reinforced concrete members should be carefully studied. Generally, the shear behavior of FRP-RC members is similar to that of steel-RC ones except for some differences attributed to the difference in properties of both materials. Differences include the modulus of elasticity, E, surface bond characteristics and the anisotropic nature of the material. Due to relatively low modulus of elasticity of FRP bars, concrete members reinforced with FRP bars develop wider and deeper cracks than members reinforced with steel having the same amount of reinforcement. Finally, the overall shear capacity of concrete members reinforced...
with FRP bars as flexural reinforcement is lower than that of concrete members reinforced with steel bars with the same amount of tensile reinforcement.

**RESEARCH SIGNIFICANCE**

With the increased popularity of using FRP bars in HSC members, a better understanding of the behaviour of such structural elements is emerging to achieve powerful and economical design methods. This study explores experimentally the effect of using HSC with FRP bars on the shear behavior of HSC beams with/without transverse shear reinforcement (stirrups). This was obtained through the experimental observations and test results related to the cracking behavior, shear failure mechanisms, ultimate shear strength $V_u$, and the midspan deflection.

**EXPERIMENTAL INVESTIGATION**

The experimental program in this paper includes thirteen HSC beams. CFRP and conventional steel bars were used in reinforcing the concrete beams. All beams have high strength concrete ($f_c = 47.07$ MPa). Beams were designed according to the ACI 440.1R-06 to fail due to shear not due to flexure. All beams were simply supported and tested under four-point monotonic loading until failure. The main investigated parameters include the shear span-to-depth ratio (a/d), the longitudinal reinforcement ratio ($\rho_f \%$), the transverse reinforcement ratio ($\rho_v \%$), and the longitudinal reinforcement type (conventional steel bars or CFRP bars).

**Materials**

Locally produced materials are used: fine aggregates composed of harsh desert sand, crushed bazalt, ordinary Portland cement, and tap drinking water. A high range water reducing admixture (Sikament R2008) was used to enhance workability and increase the early strength. Moreover, silica fume was used to get high strength concrete mixes. The target compressive strengths of the concrete were 47.1 MPa after 28 days. Table 1 gives the proportions of the mix used in this study. Six concrete cubes 150 x 150 x 150 mm were cast from each concrete batch and cured under the same conditions as the test beams. Three cubes were tested in compression after 28 days, while the other three ones were tested at the same day of testing beams. At the age of beams testing, the average compressive strength of the cubes was 58.9 MPa (equivalent to cylinder strength, $f_c = 47.1$ MPa). High-grade deformed steel bars were used as longitudinal reinforcement, while normal mild steel bars were used for stirrups. The used CFRP bars in this study were provided with ribs during manufacturing as shown in figure 1 in order to enhance the bond between the bars and concrete. CFRP bars were in two sizes (10 and 12) mm diameter. The characteristics of the CFRP and steel reinforcement used in this study are summarized in Table 2. Tests were carried out to determine the properties of the used materials according to the standard specifications, the obtained results were compared with the limits given in these specifications.

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Coarse aggregate (Bazalt size 1)</th>
<th>Coarse aggregate (Bazalt size 2)</th>
<th>Fine aggregate (sand)</th>
<th>Cement</th>
<th>Water</th>
<th>Silica fume</th>
<th>Superplasticizer (sikament R2008)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quantities</td>
<td>544 kg/m³</td>
<td>544 kg/m³</td>
<td>585 kg/m³</td>
<td>500 kg/m³</td>
<td>180 liters</td>
<td>75 kg/m³</td>
<td>14 kg/m³</td>
</tr>
</tbody>
</table>

Table 1 Concrete mix proportions

**Table 2 Properties of steel and FRP bars**

<table>
<thead>
<tr>
<th>Type of reinforcement</th>
<th>Area, (mm²)</th>
<th>Er (GPa)</th>
<th>$f_y$ (MPa)</th>
<th>$f_u$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D6 steel</td>
<td>28.2</td>
<td>200.0</td>
<td>415</td>
<td>296</td>
</tr>
<tr>
<td>D8 steel</td>
<td>50.2</td>
<td>200.0</td>
<td>443</td>
<td>353</td>
</tr>
<tr>
<td>D12 steel</td>
<td>113</td>
<td>200.0</td>
<td>540</td>
<td>392</td>
</tr>
<tr>
<td>10 mm CFRP bar</td>
<td>78.5</td>
<td>146.2</td>
<td>--</td>
<td>2130</td>
</tr>
<tr>
<td>12 mm CFRP bar</td>
<td>113.1</td>
<td>146.2</td>
<td>--</td>
<td>2130</td>
</tr>
</tbody>
</table>

Figure 1 CFRP bars used in the beams.
SPECIMEN DETAILS AND TESTING

The tested beams were arranged into four groups according to the variables. The group details are presented in Table 3. All beams had a rectangular cross section of width \(b_w=120\) mm and effective depth \(d=224\) mm. All beams had a 200 mm overhang length behind the supports on each side as anchorage length to avoid premature bond failures as presented in Figure 2. The shear span for group (1) ranged from 225 to 675, providing \(a/d\) of (1, 1.5, 2, and 3). Mild steel was used for transverse shear reinforcement for group (2) with ratios \((\rho_v\%) = 0.0, 0.22, 0.41,\) and \(0.64\). Group (3) consisted of four cases of main longitudinal reinforcement bars (Steel, CFRP, 2 CFRP +1 Steel and 1 CFRP +2 Steel). CFRP and steel were mixed in these tests in order to take the advantage of the higher stiffness of steel while keeping at the same time the higher tensile strength of CFRP. The beams of group (4) had a different longitudinal reinforcement ratio \((\rho_i = 0.584, 0.841, 1.262\) and \(1.682\%\)).

Midspan vertical deflections of beams were measured with LVDT installed at the front side at the middle of the beam. The LVDT were mounted on a metal frame clamped to the lower portion of the bottom of the beam and distributed to obtain the deflection profile of each beam. Concrete strain gages were installed at the maximum compression location of the specimen to monitor the compressive strain during testing. All instrumentations were checked and zeroed prior to the start of testing. The beams were subjected to a four-point monotonic loading up to failure at a rate of 0.5 V/min. During the tests, the development of crack patterns was marked on the beam surfaces with a black felt pen. Photographs were taken before and after failure. Loads, deflections and strains were scanned automatically until the end of the test by the data acquisition system.

![Figure 2 Beam dimensions and strain gauge positions](image)

Table 3 Details of the tested beams.

<table>
<thead>
<tr>
<th>Series</th>
<th>Beam ID</th>
<th>L_eff (mm)</th>
<th>Bar Diam (mm)</th>
<th>No. of Bars</th>
<th>shear spans a (mm)</th>
<th>a/d</th>
<th>Number of Stirrups</th>
<th>Reinforcement Type</th>
<th>Reinf Ratio (\rho_v%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Series I</td>
<td>C-1.0-V1-R2</td>
<td>950</td>
<td>12</td>
<td>2</td>
<td>225</td>
<td>1.0</td>
<td>Non</td>
<td>CFRP</td>
<td>0.841</td>
</tr>
<tr>
<td></td>
<td>C-1.5-V1-R2</td>
<td>1200</td>
<td>12</td>
<td>2</td>
<td>340</td>
<td>1.5</td>
<td>Non</td>
<td>CFRP</td>
<td>0.841</td>
</tr>
<tr>
<td></td>
<td>C-2.0-V1-R2</td>
<td>1400</td>
<td>12</td>
<td>2</td>
<td>450</td>
<td>2</td>
<td>Non</td>
<td>CFRP</td>
<td>0.841</td>
</tr>
<tr>
<td></td>
<td>C-3.0-V1-R2</td>
<td>1850</td>
<td>12</td>
<td>2</td>
<td>675</td>
<td>3</td>
<td>Non</td>
<td>CFRP</td>
<td>0.841</td>
</tr>
<tr>
<td>Series II</td>
<td>C-1.5-V1-R2</td>
<td>1200</td>
<td>12</td>
<td>2</td>
<td>340</td>
<td>1.5</td>
<td>Non</td>
<td>CFRP</td>
<td>0.841</td>
</tr>
<tr>
<td></td>
<td>C-1.5-V2-R2</td>
<td>1200</td>
<td>12</td>
<td>2</td>
<td>340</td>
<td>1.5</td>
<td>46/200mm</td>
<td>CFRP</td>
<td>0.841</td>
</tr>
<tr>
<td></td>
<td>C-1.5-V3-R2</td>
<td>1200</td>
<td>12</td>
<td>2</td>
<td>340</td>
<td>1.5</td>
<td>48/250mm</td>
<td>CFRP</td>
<td>0.841</td>
</tr>
<tr>
<td></td>
<td>C-1.5-V4-R2</td>
<td>1200</td>
<td>12</td>
<td>2</td>
<td>340</td>
<td>1.5</td>
<td>48/125mm</td>
<td>CFRP</td>
<td>0.841</td>
</tr>
<tr>
<td>Series III</td>
<td>S-1.5-V1-R2</td>
<td>1200</td>
<td>12</td>
<td>2</td>
<td>340</td>
<td>1.5</td>
<td>Non</td>
<td>Steel</td>
<td>0.841</td>
</tr>
<tr>
<td></td>
<td>C-1.5-V1-R2</td>
<td>1200</td>
<td>12</td>
<td>2</td>
<td>340</td>
<td>1.5</td>
<td>Non</td>
<td>CFRP</td>
<td>0.841</td>
</tr>
<tr>
<td></td>
<td>2C1S-1.5-V1-R3</td>
<td>1200</td>
<td>12</td>
<td>2+1</td>
<td>340</td>
<td>1.5</td>
<td>Non</td>
<td>CFRP&amp; Steel</td>
<td>1.262</td>
</tr>
<tr>
<td></td>
<td>1C2S-1.5-V1-R3</td>
<td>1200</td>
<td>12</td>
<td>1+2</td>
<td>340</td>
<td>1.5</td>
<td>Non</td>
<td>CFRP&amp; Steel</td>
<td>1.262</td>
</tr>
<tr>
<td>Series IV</td>
<td>C-1.5-V1-R1</td>
<td>1200</td>
<td>10</td>
<td>2</td>
<td>340</td>
<td>1.5</td>
<td>Non</td>
<td>CFRP</td>
<td>0.584</td>
</tr>
<tr>
<td></td>
<td>C-1.5-V1-R2</td>
<td>1200</td>
<td>12</td>
<td>2</td>
<td>340</td>
<td>1.5</td>
<td>Non</td>
<td>CFRP</td>
<td>0.841</td>
</tr>
<tr>
<td></td>
<td>C-1.5-V1-R3</td>
<td>1200</td>
<td>12</td>
<td>3</td>
<td>340</td>
<td>1.5</td>
<td>Non</td>
<td>CFRP</td>
<td>1.262</td>
</tr>
<tr>
<td></td>
<td>C-1.5-V1-R4</td>
<td>1200</td>
<td>12</td>
<td>4</td>
<td>340</td>
<td>1.5</td>
<td>Non</td>
<td>CFRP</td>
<td>1.682</td>
</tr>
</tbody>
</table>

RESULTS AND DISCUSSIONS

Figure 4 shows the applied load versus midspan deflection responses, while Table 4 summarizes the flexural cracking load \(P_{cr}\), load at shear failure \(P_u\), major diagonal cracking shear \(V_{cr}\), ultimate shear strength \(V_u\),
Midspan deflection at failure ($\Delta_{\text{max}}$), Slope angle of the inclined failure crack ($\theta$), and the failure mode for all beams.

Table 4 Summary of test results

<table>
<thead>
<tr>
<th>Series</th>
<th>Beam ID</th>
<th>$P_{\text{cr}}$ (ton)</th>
<th>$P_{\text{f}}$ (ton)</th>
<th>$V_{\text{cr}}$ (ton)</th>
<th>$V_{\text{f}}$ (ton)</th>
<th>$\Delta_{\text{max}}$ (mm)</th>
<th>$\theta$ (deg)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>C-1.0-V1-R2</td>
<td>6</td>
<td>23</td>
<td>11</td>
<td>11.5</td>
<td>1.98</td>
<td>59*</td>
<td>S.C</td>
</tr>
<tr>
<td>II</td>
<td>C-1.5-V1-R2</td>
<td>4.1</td>
<td>22.3</td>
<td>7.7</td>
<td>11.15</td>
<td>7.99</td>
<td>46*</td>
<td>D.T</td>
</tr>
<tr>
<td>III</td>
<td>C-1.5-V2-R2</td>
<td>3.4</td>
<td>24.5</td>
<td>8.4</td>
<td>12.625</td>
<td>10.00</td>
<td>44*</td>
<td>D.T</td>
</tr>
<tr>
<td>IV</td>
<td>C-1.5-V3-R2</td>
<td>2.9</td>
<td>25.3</td>
<td>10.2</td>
<td>12.675</td>
<td>10.91</td>
<td>43*</td>
<td>D.T</td>
</tr>
<tr>
<td>V</td>
<td>C-1.5-V4-R2</td>
<td>2.4</td>
<td>26.9</td>
<td>14.1</td>
<td>13.45</td>
<td>11.33</td>
<td>44*</td>
<td>D.T</td>
</tr>
<tr>
<td>Series</td>
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<td>4.1</td>
<td>22.3</td>
<td>7.7</td>
<td>11.15</td>
<td>7.99</td>
<td>46*</td>
<td>D.T</td>
</tr>
<tr>
<td>III</td>
<td>C-1.5-V1-R3</td>
<td>5</td>
<td>22.8</td>
<td>14</td>
<td>11.4</td>
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<td>33*</td>
<td>D.T</td>
</tr>
<tr>
<td>Series</td>
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<td>3.4</td>
<td>18.1</td>
<td>6</td>
<td>9.05</td>
<td>7.36</td>
<td>44*</td>
<td>S.C</td>
</tr>
<tr>
<td>IV</td>
<td>C-1.5-V1-R4</td>
<td>4.8</td>
<td>25.2</td>
<td>7.4</td>
<td>12.6</td>
<td>8.65</td>
<td>47*</td>
<td>D.T.B</td>
</tr>
</tbody>
</table>

Crack Patterns and Modes of Failure

During the test, the growth of cracks was marked for each beam. This was carried out to identify the direction of cracks propagation and to determine the differences in cracks patterns of the beams. The slope of the inclined crack at failure is listed in Table (4). For all beams, the first flexural cracks initiated at the bottom of the beam in the constant moment region, where the flexural tension stress was maximum and the shear stress was zero. The observed flexural cracks propagated vertically upward to the level of the neutral axis, which reflected the absence of shear stresses. As the load increased, additional flexural cracks developed within the shear span. Due to the presence of shear stresses, these flexural cracks became progressively more inclined and propagated towards the load points. These types of cracks are known as flexural-shear cracks. These cracks extended rapidly through the beam leading to the so-called a diagonal-tension failure. The duration between the formation of an inclined crack and failure of a beam was small. This behavior was observed for most of the beams in the current study. For beams with different a/d, the slope of the inclined crack decreased as a/d increased. This is attributed to that at a certain shear load, the moment as well as the flexural stress increase as a/d increases. Higher flexural stress could lead to the reduction in the inclination of shear cracks. Hence, the horizontal projection of the inclined cracks increased with an increase in a/d. The loads of initial flexural cracks and initial shear cracks decrease as a/d increases. The cracking and the ultimate loads increase as a/d decreases. The increase of steel stirrups ratio ($\rho_v$%) enhances the ductility of beams and changes the mode of failure from brittle failure to ductile one. A secondary concrete splitting failure within the shear span at the level of the longitudinal reinforcement was observed in beam "1C2S-1.5-V1-R3". This phenomenon of longitudinal splitting along the main reinforcement occurred immediately after the formation of the critical diagonal shear crack. Due to the smoother surface of the shear crack of HSC beams, the
contribution of aggregate interlock in transferring shear stresses becomes less significant resulting in higher dowel forces in the longitudinal reinforcing bars crossing the crack. These higher dowel forces cause higher vertical tensile stresses in the concrete surrounding the bars. These tensile stresses combined with the existing bond stresses result in concrete splitting failure along the longitudinal bars.

**Cracking and Ultimate Loads**

Figure 4 and Table 4 show that the ultimate shear capacity increased significantly with decreasing a/d. It can be noticed from the figures that the increase in a/d from 1.0 in beam "C-1.0-V1-R2" to 3.0 in beam "C-3.0-V1-R2" caused a reduction in the ultimate shear capacity by approximately 72%. The decrease in the ultimate shear strength with increasing a/d can be attributed to the fact that the tied arching action becomes less effective with increasing a/d because of the reduced angle between the inclined strut and longitudinal axis of the beam. The decrease in a/d reduces the distance between the supports and the applied loads and hence increases the effectiveness of the arching mechanism by transmitting a greater part of load directly to the support by diagonal compression. This was confirmed by many authors (Razaqpur, et al., 2004, El-Sayed, et al., 2006a, and Razaqpur, et al., 2011). From Table 4 and Figure 4 it is observed that the values of the ultimate shear strength increased linearly as ρv% increased. This increase in the shear strength was due to the improvement in the shear transfer mechanisms. It was unexpected to observe that the ultimate shear capacity of the beam reinforced with steel were slightly lower than that of the counterpart reinforced with CFRP bars as shown in Figure 4 and Table 4. This result contradicts the general expectation, that due to the relatively low modulus of elasticity of FRP bars, the FRP-reinforced concrete beams experience reduced shear strength in comparison to those of beams reinforced with the same amount of conventional steel bars. In fact, the modulus of elasticity was not the only difference between the steel and FRP bars used in this investigation. There was another parameter which may cause this result. This parameter was the difference between the tension strength of CFRP and conventional steel bars. As a result, the arch action developed in the FRP-reinforced beams was more efficient than that developed in the steel-reinforced beams. The more efficient the arch action is, the higher the ultimate shear strength will be. The efficient arch action in the FRP-reinforced beams was to the extent to overcome the difference in the modulus of elasticity between FRP and steel bars, resulting in slightly higher ultimate shear strengths in the beams reinforced with CFRP bars than those of the beams reinforced with steel. The increase in reinforcement ratio by approximately 188% (from 0.584 to 1.682%) increased the ultimate shear strength by 43%. This increase in shear strength was due to the improvement in the shear transfer mechanisms. Increasing the reinforcement ratio decreased the penetration depth in the compression zone of the shear cracks and decreased their width. This, in turn, increases the contribution of aggregate interlock as well as the contribution of un-cracked concrete by increasing the area of concrete compression zone. In addition, increasing the reinforcement ratio increased the dowel capacity of the member by increasing the dowel area, thereby reduced the tensile stresses induced in the surrounding concrete. This was in agreement with a previous work by El-Sayed, 2006b

**Load Deflection Behavior**

The measured values of midspan deflection at the bottom surface of the tested beams were plotted against the corresponding applied load from zero to failure as shown in Figure 2. Table 4 gives also the values of the midspan deflection at failure for each beam. It is observed that the shear load deflection behavior of all tested beams was linear. This linear behavior continued until failure because of the stiffness of the beams remained constant while the formed cracks grew and new cracks developed in the shear span zone. This is due to the linear elastic characteristic of CFRP bars. It is observed that, decreasing a/d in beams from 3.0 to 1.0 led to a reduction in the maximum midspan deflection from 9.5 mm to 4.0 mm. Also, the lower the a/d, the steeper the load-deflection curve. This may be attributed to the fact that when a/d decreases, the beam becomes more rigid. It is observed also that the cracking and ultimate load values and the maximum deflection decrease as ρv% decreases. This means that as the stirrups increased in the beam, the ductility of that beam increases. At the same load level, and as expected, the deflection of the beams decreased as the axial stiffness of the reinforcing bars increased, irrespective of the reinforcement type.

**CONCLUSIONS**

The present study investigated the shear behavior of HSC beams reinforced with CFRP bars. The experimental study carried out included testing thirteen high strength concrete beams to investigate the factors affecting the shear behavior of HSC beams reinforced with FRP bars. The major test variables were: the shear span to depth ratio, (a/d); the longitudinal reinforcement Ratio (ρl %); the shear reinforcement ratio (ρv %) and longitudinal reinforcement type (CFRP and High tensile steel). Based on the experimental observations and the analysis of the test results, the following conclusions were drawn:
1) The increase in a/d from 1.0 to 3.0 led to a reduction in the ultimate shear capacity by approximately 72%.
2) For beams with different a/d, the slope of the inclined crack decreased as a/d of the beam increased.
3) The increase in $\rho_v$% enhances the ductility of beams and changes the mode of failure from brittle failure to ductile one.
4) It was unexpected to observe that the ultimate shear capacity of the beam reinforced with steel bars were slightly lower than that of its counterpart reinforced with CFRP bars. This may be attributed to the higher tensile strength of CFRP bars.
5) The increase in reinforcement ratio by approximately 188% (from 0.584 to 1.682%) increased the ultimate shear strength by 43%.
6) Using steel bars at the same time with CFRP bars as longitudinal reinforcement enhances the ultimate shear capacity of the tested beams and enhances their ductility.
7) The decrease in a/d from 3.0 to 1.0 led to a reduction in the maximum midspan deflection from 9.5 mm to 4.0 mm. Moreover, the lower the a/d, the steeper the load-deflection curve.
8) As $\rho_v$% increases, the ductility of the beams increases.
9) As expected, the behavior of beams with steel reinforcement was different from the other beams that reinforced with CFRP. A yield stage occurred in beams with steel reinforcement due to the yielding of steel reinforcement, while in beams that reinforced with CFRP the load deflection curve was linear. This is due to the linear elastic characteristic of CFRP bars.
10) Generally, HSC beams reinforced with CFRP bars can achieve shear strength values comparable to or higher than that of similar steel-reinforced beams.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the sincere assistance provided by Dr. Mohamed Fathy Fahmy, the assistant professor at Assiut University, Egypt (Currently, a research Fellow, International Institute for Urban Systems Engineering, Southeast Univ, Nanjing, China). Dr. Mohamed's effort in importing CFRP bars from China is highly appreciated.

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STUDY OF THE STRUCTURAL BEHAVIOUR OF ECC FOR LINK SLABS

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⁵ Department of Civil Engineering, Dongguan University of Technology, China

ABSTRACT

With the increasing traffic loading and large variation of environmental temperatures, deteriorations of expansion joints between simply supported spans cause structural repeated maintenance and high repair costs. The focus of this study is to study the behaviour of a link slab element, which can be used as a joint between two adjacent structural components. In this joint system, a ductile Engineering Cementations Composites (ECC) link slab reinforced with Fibre Reinforced Polymer (FRP) grid is proposed for replacing the conventional expansion joint. The requirements on the link slab in terms of material properties and reinforcement with FRP grid. This research herein describes the study of mechanical performance of different mixtures ECC specimens and the mechanical properties of FRP grid. The experimental variables include the volume of different type powder, sand ratio, water-cement ratio and tensile strength of FRP grid. Subsequently, the test results of the flexural behaviours of reinforced ECC test specimens were presented and discussed. It was shown the ductility and cracking behaviour of ECC.

KEYWORDS

ECC, FRP grid, link slab, mixing procedure, ductility.

INTRODUCTION

Large scale highway and superhighway infrastructure remains a backbone of national and international trade supporting the economies of both highly developed and developing nations worldwide. However, to allow the condition of infrastructure is allowed in many continue to become exceedingly poor, mainly due to a persistent lack of funding, increasing traffic volumes, and heavier loads on road ways (Lepech M.D. 2009). Many highway bridges are composed of multiple span steel girders simply supported at piers or bents. A mechanical joint is typically employed at the end of the simple span to allow deck deformations imposed by girder deflection, concrete shrinkage, and temperature variations (Yun Y.K. 2004). Many infrastructure maintenance and repair meth-odds have been proposed and instituted, ranging from the use of high strength concrete (Kim et al.2001) to the use of epoxy coated reinforcing steel (Zahrani et al.2002) on bridge decks, each with varying degrees of success. However, none of these solutions target the inherent shortfall of concrete brittleness, which results in cracking with the applied load. The crack width is difficult to be controlled in a reliable manner, typically allow salt water to contact the reinforcing steel, could cause series structural deterioration subsequently. It is well known that it is costly to install and maintain bridge deck joints (Wolde-Tinsae et al.1987). Deterioration of joint functionality due to debris accumulation can lead to severe damage in the bridge deck and substructure. Alampalli and Yannotti found that the jointless deck construction practice is generally more efficient than integral bridge construction practice (Alampalli, S et al.1998).

Engineered Cementitious Composite (ECC) is a high performance fiber reinforced cementitious composite designed to resist tensile and shear force while retaining compatibility with normal concrete in almost all other respects (Li. 2002). Recent research on ECC has shown that this material is highly durable and well suitable for
large infrastructure applications. After first cracking, the composite undergoes plastic yielding and strain-hardening to a tensile strain of 3.7% prior to developing a macroscopic crack. The tensile strain capacity of ECC is about 380 times that of normal concrete. Especially, crack width of ECC material develops between 50μm and 70μm during early strain hardening stages and remain at that width under additional tensile strain up to failure. The goal of this research is to provide a cost-effective solution to bridge deck deterioration problems associated with joints, by developing durable and maintenance-free ECC link slabs used in jointless bridge decks. Thus, the introduction of ECC to link slab construction is proposed for its ability to control crack widths and its processing flexibility. Link slab of ECC reinforced with FRP grid is shown in Figure 1.

![Figure 1](image)

Figure 1 Link slab of ECC reinforced with FRP grid

This research herein describes the study of mechanical performance of various ECC specimens with different mixtures, mechanical property tests of CFRP grid and flexural properties of slabs. The experimental of bending variables include the volume of different type of powder, water-cement ratio, the mechanical property of FRP grid. In this article, the ECC reinforced with FRP grid was discussed through bending tests to find out advantages of the combination of these two materials.

**EXPERIMENTAL PROGRAMME**

**Properties of PVA Fiber and Tensile Test of FRP Grid**

Large scale laboratory testing of ECC link slabs was conducted by Kim et al. (2001) to investigate the load capacity and fatigue performance of ECC link slabs, along with the development of cracking on the tensile face of the ECC link slab (Wang et al. 1997). Kim found that ECC material was a suitable choice for construction of link slabs to replace conventional mechanical expansion joints. In order to obtain higher corrosion resistance and durability, polyvinyl alcohol (PVA) fiber was used in this article. It was found that the high modulus polyvinyl alcohol (PVA) fiber have remarkable effect on the improving strain-capacity of ECC than polypropylene (PP) fiber (HR Pakravan et al. 2016). The fibers are produced by the company of Kuraray and properties of PVA fiber are shown in Table 1.

<table>
<thead>
<tr>
<th>Type of fiber</th>
<th>Length /mm</th>
<th>Diameter /um</th>
<th>Tensile strength /MPa</th>
<th>Young’s modulus /Gpa</th>
</tr>
</thead>
<tbody>
<tr>
<td>PVA fiber</td>
<td>12</td>
<td>31</td>
<td>1600</td>
<td>47.29</td>
</tr>
</tbody>
</table>

![Table 1 Properties of PVA fiber](image)

![Figure 2](image)

Figure 2 FRP Grid

![Figure 3](image)

Figure 3 Tensile Test
In order to develop new type of anti-cracking technique for ECC link slabs, a new cementitious composite reinforced by a combination of carbon textile and PVA fiber was proposed. FRP grid was placed along the direction of principal stress so that the reinforcing effect can be significantly enhanced. The FRP grid used in this test was shown as Figure 2. The tensile behaviour of this FRP grid was obtained by the direct tensile test (see Figure 4). The maximum tensile strength of the FRP grid is 1.56 N/mm² according to the test results.

**Mixing Process and Mix Proportions of ECC Specimens**

The dispersion of fiber is one critical factor for the workability of ECC. The mixing process of ECC in this paper is illustrated in Figure 2, which is beneficial for the dispersion of fiber. In the process of mixing, adding superplasticizer and water should be in advance to make sure the degree of dispersion of PVA fiber. Thereafter, PVA fiber was added evenly into the cement-sand mortar to make sure the magnitude of the stirring.

![Figure 4 Mixing process of this test](image)

Based on the property requirements of ECC material for link slabs, a number of mix proportions were adopted based on the design criteria, as shown in Table 2. The experimental variables include the volume of different type powder, sand ratio and water-cement ratio. At the same time, a control group of concrete was set up. The compressive strength is 35MPa.

<table>
<thead>
<tr>
<th>Set</th>
<th>Powders</th>
<th>Water binder ratio</th>
<th>Sand binder ratio</th>
<th>Superplasticizer (%)</th>
<th>PVA Fiber (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.5</td>
<td>0.5</td>
<td>0.36</td>
<td>0.1</td>
<td>2.0</td>
</tr>
<tr>
<td>2</td>
<td>0.5</td>
<td>0.45</td>
<td>0.36</td>
<td>0.1</td>
<td>2.0</td>
</tr>
<tr>
<td>3</td>
<td>0.5</td>
<td>0.4</td>
<td>0.36</td>
<td>0.1</td>
<td>2.0</td>
</tr>
<tr>
<td>4</td>
<td>0.5</td>
<td>0.45</td>
<td>0.36</td>
<td>0.1</td>
<td>2.0</td>
</tr>
<tr>
<td>5</td>
<td>0.5</td>
<td>0.4</td>
<td>0.36</td>
<td>0.1</td>
<td>2.0</td>
</tr>
<tr>
<td>6</td>
<td>0.4</td>
<td>0.6</td>
<td>0.36</td>
<td>0.1</td>
<td>2.0</td>
</tr>
<tr>
<td>7</td>
<td>0.3</td>
<td>0.7</td>
<td>0.36</td>
<td>0.1</td>
<td>2.0</td>
</tr>
</tbody>
</table>

![Figure 5 Steel framework of ECC slabs](image)  
![Figure 6 Three-point loading test](image)

**Specimen Preparation and Bending Test**

According to the previous experimental studies (Alp Caner et al. 1998), the link slab is in bending and behaved similar as a flexural component rather than a tension member and the crack at the center of slab did not extend to
the bottom face of the slab. In order to analyze the bending performance of link slab, a series of ECC slabs were designed and tested. The size of ECC slab was determined to be 15mm × 100mm × 400mm (see Figure 5) based on the research in the literatures. In the study, a three-point loading test was carried out to investigate the bending behaviour of all the ECC slabs according to the ASTM code (see Figure 6).

**DISCUSSION OF EXPERIMENTAL RESULTS**

**Flexural Properties of Concrete and ECC Slabs**

The bending test results show that ECC slab possesses ultra high flexural ductility, as shown in Figure 7, and have saturated multiple-cracking capability than concrete slab. The mid-span deflection of ECC slab is 133 times than concrete slab (see Figure 8). In addition, the volume of limestone flour has an effect on the deflection of mid-span. Increasing 10% limestone flour in the mixing resulted in 18.8% enhancement of the flexural capacity of ECC slab (see Figure 9). On the other side, as shown in Figure 10, the use of silica fume result in an increase of the cracking load by 46.3% and improve the cracking behaviour of pure bending zone, but declined the flexural capacity by 9.5%. Therefore, it can be summarised that the application of mineral powder (limestone and Silica) has a significant effect on the ductility of ECC materials, which is also presented by other researchers (Shuxin Wang et al. 2007). It is necessary to develop a further study of the influence of mineral powder on the micro-crack and bending capacity of ECC link slab.

**Influence on FRP Grid on the Behaviour of ECC Slabs**

Figure 11 illustrates the comparison of load vs. deflection response of the ECC slab reinforced with FRP grid in the middle layer and that without any reinforcement. It is found that the increase of flexural capacity of the reinforced ECC with FRP grid is 1.7% (see Figure 11). As shown in Figure 12, ECC reinforced with FRP grid can improve the cracking behaviour of ECC, cracks of slabs become finer and the spaces between closer. In addition, it is shown that average crack width of reinforced ECC slab is 78.8μm in pure bending zone, while the average crack width of ECC slab is 118.2μm (see Figure 13). It is found that the using of FRP grid as reinforcing material resulted in increasing the corrosion resistance and bending stiffness of ECC component.
CONCLUSION

This study is to investigate the potential of a durable concrete bridge deck system with the use of ECC material in link slabs. These trial mixes confirmed that large scale mixing of ECC material is possible and could result in a material that maintained its high performance in large quantity processing with conventional ready-mix equipment. According to the result of experiments, mineral powders determine the bending capacity and the micro crack develop of ECC slabs. It is evident in the test results that the application of mineral powder (limestone and Silica) has a significant effect on the ductility of ECC materials, increasing 10% limestone flour in the mixing resulted in 18.8% enhancement of the flexural capacity of ECC slab. ECC with high volume limestone flour have better work performance compared to that with small volume limestone flour. The use of silica fume could increase the crack load by 46.3% and improve the cracking behaviour of pure bending zone, but decline the flexural capacity by 9.5%. ECC slab reinforced with FRP grid results in better cracking behaviour observably, but have finite influence on ductility. The application of FRP grid as a reinforcing material in ECC slabs has a beneficial effect on the structural behaviour of this structural component.

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REUSE OF WASTE GFRP COMPOSITES AS COARSE AGGREGATES FOR CONCRETE

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City College of New York
New York, NY, USA

ABSTRACT

The use of production waste FRP fully-cured parts (not scrap fibers or resin systems) as replacements for large (coarse) aggregate in concrete mixes is discussed in this paper. A preliminary study consisting of two series of tests using production waste pulltruded FRP reinforcing bars were conducted. FRP Bars ranging from 6 mm to 25 mm in diameter were cut into cylindrical aggregate-sized pieces and used as a replacement for the coarse aggregate in normal strength (~30 MPa) and high strength (~40 MPa) mixes in natural aggregate replacement percentages of 5, 10, 40 and 100%. Mixes were graded according to ASTM standards. 100 × 200 mm and 150 × 300 mm standard test cylinders were produced and tested for compressive strength, tensile (splitting) strength, and compressive stiffness. Strength and stiffness data are presented and compared with ACI and EU2 predictions. Images of failure modes and failure surfaces as a function of the replacement percentages are provided. The significance of these results on the possible use of aggregate pieces obtained by processing GFRP waste from FRP wind turbine blades and FRP boats are discussed.

KEYWORDS

FRP waste, FRP recycled aggregates, FRP rebars, concrete strength, concrete stiffness.

INTRODUCTION

In recent years there has been an increasing body of research, and industry concern, related to recycling or reuse of FRP composites (Job, 2013; Bank and Yazdanbakhsh, 2014; Oliveux et al., 2015). Three primary methods to dispose of FRP composites exist at the present time, (1) disposal in a landfill, (2) incineration, and, (3) reusing all or part of the composite material in a secondary process or application. Methods (1) and (2) are already prohibited in a number of countries. Reuse can consist of (a) recovery of the constituent fibers or the resins by thermo–chemical methods, (b) use of small pieces or particles of FRP material itself by cutting and grinding, or (c) use of large parts or even the entire FRP part in a different application. This paper focuses on method (b) and exclusively on glass fiber reinforced thermosetting polymers (GFRP).

In the near future significant FRP waste will come from the wind power industry (Cherrington et al., 2012, Ortegon et al., 2013) and the marine industry (Marsh, 2013). The rapid growth in wind energy technology in the last 15 years has led to a commensurate rapid growth in the amount of FRP materials used in this industry. The Global Wind Energy Council (GWEC) reports that 432 GW of wind power was installed as of the end of 2015; China 145 GW, USA 74 and Germany 44 leading the world. One wind blade of a typical 2.5 MW turbine is 50 m long, contains approximately 8 tonnes of FRP material, and costs about $150,000. By 2035, 4.2 million tonnes of wind blade (based on a 2015 installed capacity of 432 GW of wind power) will need to be recycled.

In this study GFRP waste from commercially produced GRFP reinforcing bars for concrete structural members were used as coarse aggregate for structural concrete as a replacement for natural aggregate. The use of FRP recyclates in concrete as ground filler has been studied for a number of years (Yazdanbakhsh and Bank, 2014). However, very little work has been reported on the use of large (5 mm to 25 mm) aggregates from recycled FRP in concrete. The benefits of using larger FRP aggregates from FRP waste are hypothesized by the authors to be due to (1) the ability to grade the aggregates when they are cut to specific sizes, (2) the ability to preserve the composite nature of the FRP which is lost when it is ground to a fine power to be used as filler, (3) the ability to save on energy cost when reducing FRP waste parts to larger pieces, and, (4) the ability to reduce environmental
pollution due to pulverizing or incinerating the FRP material. The reported study represents the initial phase of this work in which waste FRP reinforcing bars were used as coarse aggregates. Current work using waste FRP in different shapes and sizes from end–of–life wind turbine blades, boats and pultruded profiles is underway.

**EXPERIMENTAL INVESTIGATION**

**Mix Designs**

Two separate series of test programs were conducted. Series 1 consider of concrete mixes with high percentages (100, 40) of FRP Recycled aggregate (FRP–RA) and Series 2 consider of concrete mixes with low percentages of FRP–RA. In Series 1 both high strength and normal strength concrete mixes were considered. The mix designs are shown in Table 1. Note that different compositions (sizes of aggregates) of the FRP–RA were used in different mixes to investigate different aspects of behavior in this preliminary study.

**Table 1 Concrete mix designs for test specimens**

<table>
<thead>
<tr>
<th>Concrete mix</th>
<th>FRP-RA vol. replacement ratio, %</th>
<th>w/c</th>
<th>Total agg. (coarse and fine) /concrete vol. ratio</th>
<th>Coarse agg./total agg. vol. ratio</th>
<th>Coarse agg./concrete vol. ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>NC1</td>
<td>0</td>
<td>0.57</td>
<td>0.70</td>
<td>0.55</td>
<td>0.39</td>
</tr>
<tr>
<td>N40</td>
<td>40*</td>
<td>0.57</td>
<td>0.70</td>
<td>0.55</td>
<td>0.39</td>
</tr>
<tr>
<td>N100</td>
<td>100**</td>
<td>0.57</td>
<td>0.70</td>
<td>0.55</td>
<td>0.39</td>
</tr>
<tr>
<td>H01</td>
<td>0</td>
<td>0.44</td>
<td>0.60</td>
<td>0.67</td>
<td>0.40</td>
</tr>
<tr>
<td>H40</td>
<td>40*</td>
<td>0.44</td>
<td>0.60</td>
<td>0.67</td>
<td>0.40</td>
</tr>
<tr>
<td>H100</td>
<td>100**</td>
<td>0.44</td>
<td>0.60</td>
<td>0.67</td>
<td>0.40</td>
</tr>
<tr>
<td>NC2</td>
<td>0</td>
<td>0.45</td>
<td>0.61</td>
<td>0.58</td>
<td>0.35</td>
</tr>
<tr>
<td>N05</td>
<td>5***</td>
<td>0.45</td>
<td>0.61</td>
<td>0.58</td>
<td>0.35</td>
</tr>
<tr>
<td>N10</td>
<td>10***</td>
<td>0.45</td>
<td>0.61</td>
<td>0.58</td>
<td>0.35</td>
</tr>
</tbody>
</table>

**NOTES:** NS: Normal Strength. HS: High Strength
*only ⅛" (19 mm) and 1" (25 mm) size aggregates replaced with FRP–RA
** ⅛" (6 mm), 3/8" (10 mm), ½" (12 mm), 5/8" (16 mm), ⅜"(19mm), and 1" (25mm) replaced with FRP–RA.
*** ¼"(6 mm), 3/8" (10 mm), ½" (12 mm) and ¾"(19mm) replaced with FRP–RA. 1" (25 mm) natural aggregate NOT used.

The FRP–RA were cut from lengths of waste Aslan™ FRP rebars ranging from No.2 (1/4 inch (~ 6 mm) to 1 inch (~25mm)). The pieces had lengths equal to their diameters as shown in Figure 1. The control mixes were cast using natural aggregates which were crushed granite. A side–by–side view of the 100 % NA and 100% FRP–RA is shown Figure 2. Further details of the mix design procedures following ASTM standards are provided in Yazdanbakhsh et al., 2016.

![Figure 1 FRP–RA aggregate sizes and aspect ratios](image)
For each series both compression and splitting tension tests were conducted to measure strengths and in Series 2 (only) stiffness. Typical compression and splitting test configurations shown in Figure 3.

**Test Results**

Compression and splitting test results for the two series are shown in Table 2. Splitting test results Compressive strength results were compared with predictions from the ACI and Eurocode 2 which allow splitting strength to be calculated from measured compressive strength according to equations (1) and (2) below.

\[
f_{ct,ACI} = 0.56 f_{cm}^{0.5} \quad \text{(MPa)} \\
\]

\[
f_{ct,EU2} = 0.33 f_{cm}^{0.67} \quad \text{(MPa)}
\]

<table>
<thead>
<tr>
<th>Mix</th>
<th>( f'_c ) (MPa)</th>
<th>% decrease from NC</th>
<th>COV ( f'_c )</th>
<th>( f_{ct} ) (MPa)</th>
<th>% decrease from NC</th>
<th>COV ( f_{ct} )</th>
<th>( f_{ct,ACI} ) (MPa)</th>
<th>( f_{ct,EU2} ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NC1</td>
<td>37.5</td>
<td>-</td>
<td>3.8</td>
<td>4.0</td>
<td>-</td>
<td>6.4</td>
<td>3.43</td>
<td>3.19</td>
</tr>
<tr>
<td>N40</td>
<td>32.8</td>
<td>-13</td>
<td>1.0</td>
<td>3.0</td>
<td>-25</td>
<td>10.2</td>
<td>3.21</td>
<td>2.83</td>
</tr>
<tr>
<td>N100</td>
<td>29.5</td>
<td>-21</td>
<td>2.4</td>
<td>2.6</td>
<td>-35</td>
<td>5.4</td>
<td>3.04</td>
<td>2.58</td>
</tr>
<tr>
<td>HC1</td>
<td>46.3</td>
<td>-</td>
<td>5.5</td>
<td>4.5</td>
<td>-</td>
<td>5.3</td>
<td>3.81</td>
<td>3.79</td>
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<tr>
<td>H40</td>
<td>40.4</td>
<td>-13</td>
<td>4.9</td>
<td>4.0</td>
<td>-11</td>
<td>5.2</td>
<td>3.56</td>
<td>3.39</td>
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<tr>
<td>H100</td>
<td>36.6</td>
<td>-21</td>
<td>6.1</td>
<td>3.6</td>
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<td>5.3</td>
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</table>

<table>
<thead>
<tr>
<th>Mix</th>
<th>( f'_c ) (MPa)</th>
<th>% decrease from NC</th>
<th>COV ( f'_c )</th>
<th>( f_{ct} ) (MPa)</th>
<th>% decrease from NC</th>
<th>COV ( f_{ct} )</th>
<th>( f_{ct,ACI} ) (MPa)</th>
<th>( f_{ct,EU2} ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NC2</td>
<td>40.2</td>
<td>-</td>
<td>2.2</td>
<td>3.4</td>
<td>-</td>
<td>4.7</td>
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<td>3.92</td>
</tr>
<tr>
<td>N05</td>
<td>37.9</td>
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<tr>
<td>N10</td>
<td>38.9</td>
<td>-3</td>
<td>2.2</td>
<td>3.4</td>
<td>0</td>
<td>2.1</td>
<td>3.49</td>
<td>3.84</td>
</tr>
</tbody>
</table>

The results show that in Series 1 the compressive strength of the FRP–RA specimens is reduced by 13 to 21% relative to the NA control for both the normal and high strength concrete, while the splitting strength is reduced by 25 to 35% for the normal strength mix and only 11 to 20% for the high strength mix. In Series 2 the
Compressive strength reduction was between 3 and 6% and the splitting strength between 9 and 0%. Counter to expectations the reductions were larger for the 5% replacement than the 10% replacement which casts doubts on the integrity of the Series 2 results. In Table 2 the predicted splitting strengths shown in green color indicate that the measured strength is conservative while the red color indicates an unconservative prediction. It can be seen that Series 2 results were unconservative even for the NA mix designs. This also raises some question about the integrity of the results of series 2. Notwithstanding the results of Series 2 are reported since the mix proportions were quite different from Series 1 and the cylinder sizes were bigger so the results of are some interest. Images of the failed compression test specimens are shown in Figs. 4 to 6.

In Series 2 strain gages were bonded to the specimens to measure the compressive modulus of the specimens. The results are shown in Table 3 with comparison to the ACI predictions.
Table 3 Stiffness from Compression Test
(Strain gage measurement per ASTM 469)

<table>
<thead>
<tr>
<th>Mix</th>
<th>E (GPa)</th>
<th>$E_{ACI}$ (GPa)</th>
<th>% diff</th>
</tr>
</thead>
<tbody>
<tr>
<td>NC2</td>
<td>26.6</td>
<td>29.8</td>
<td>-12.0</td>
</tr>
<tr>
<td>N05</td>
<td>29.5</td>
<td>28.9</td>
<td>+2.0</td>
</tr>
<tr>
<td>N10</td>
<td>33.4</td>
<td>29.3</td>
<td>+12.3</td>
</tr>
</tbody>
</table>

From Table 3 it is seen that the stiffness for the control mix (NC2) in Series 2 was less than the ACI prediction for compressive stiffness calculated from the measured compressive strength given in equation 3. The predictions for the FRP–RA mixes were conservative (and larger than the control mix). It appears the control mix was not well made.

$$E_{ACI} = 4,700 \sqrt{f'_c} \text{ (MPa)}$$  \hspace{1cm} (3)

Images of the splitting specimens are shown in Figs. 7 to 9 below. The darker color of the Series 2 specimens is due to the fact that they were wet before the photographs to provide visual contrast.

Figure 7 Series 1, splitting tests, normal strength concrete (N100 shown enlarged on the right)

Figure 8 Series 1, splitting tests, high strength concrete (H100 shown enlarged on the right)

Figure 9 Series 2, splitting tests, normal strength concrete (N05 shown enlarged on the right)

**DISCUSSION**

Detailed discussion of the test results for Series 1 are presented in Yazdanbakhsh et al., 2016. The reduced strength in the concrete with the FRP–RA is attributed to the bond in the interfacial transition zone (ITZ).
between the FRP–RA and the cement paste. Although the test results show a decrease in the compressive and the splitting strength when FRP-RA is substituted for NA in the specimens this is in a similar range, in many reported tests, to the decrease in strength seen when NA is replaced with conventional Recycled Concrete Aggregate (RCA) obtained from construction and demolition waste (CDW). In these series of tests, FRP reinforcing bars with helical wraps and sand-coated longitudinal surfaces, which are especially produced for developing bond to the concrete, were used. In cases where boat parts or wind blade parts need to be cut up and graded to produce FRP-RA all the surfaces are likely to be smooth. Engineering solutions to resolve this design issue will need to be developed in the future. Finally it is also important to point out that this research has not yet addressed the subject of the durability of the FRP–RA concrete.

ACKNOWLEDGMENTS

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ABSTRACT

Carbon Fibre Reinforced Polymer (CFRP) is an advanced composite material with advantages of high strength, lightweight, no corrosion and high fatigue resistance, which makes it suitable to be made into cables and replace steel cables in cable structures. However, anchoring CFRP cables is problematic, because CFRP is an orthotropic material, whose strength and modulus perpendicular to the fibre direction are considerably smaller than those in the fibre direction. To avoid the anchoring problem, a novel design, i.e. CFRP continuous winded band system, is proposed in this paper. Cables used in such system are CFRP continuous bands, which will be wound through all the intermediate nodes and only anchored at the both end nodes or which will form closed loops without any anchorage. The greatest advantage of the CFRP continuous winded band system is minimising necessary anchorages and thus exploiting to the full favourable conditions of CFRP cables and avoiding the unfavourable ones.

KEYWORDS

CFRP, cable, anchorage, continuous band, winding.

INTRODUCTION

Reviewing the history of cable structures, it can be found that the development of cable materials can significantly promote the development of structures (Liu 2015). Carbon Fibre Reinforced Polymer (CFRP) is an advanced composite material with advantages of high strength, lightweight, no corrosion and high fatigue resistance, which makes it suitable to be made into cables and replace steel cables in a broad range of applications (Schlaich et al. 2015).

The application of CFRP cables is now driving the progress of cable structures (Liu 2015). However, the CFRP is a typical orthotropic material, that is, its mechanical properties, such as the strength and the modulus, perpendicular to the fibre direction is considerably lower than those in the fibre direction. This makes CFRP cables difficult to be anchored.

This paper proposes a novel design, i.e. CFRP continuous winded band system, so as to avoid the anchorage problem of CFRP cables. The CFRP continuous bands are adopted in this system. They are wound through all the intermediate nodes and only anchored at the both end nodes or form closed loops without any anchorage. Several possible forms of such system are illustrated in this paper. In addition, state of the art and theoretical basis are first introduced.

STATE OF THE ART

Existing CFRP Cable Structures

Studies on CFRP cable structures can be dated back to the 1980s (Meier 2012). As early as in 1987, a CFRP cable-stayed bridge with a main span of 8400 m crossing the Strait of Gibraltar was proposed (Meier 1987). However, the first practical use of CFRP cables in a real cable structure was in 1996 in the Tsukuba FRP Bridge (Karbhari 1998). From then to now, there have been 10 CFRP cable structures over the world, even though all of them were built more or less as prototypes. The existing CFRP cable structures are listed in chronological order of completion in Figure 1 (Liu 2015).
In the above figure, No. 1 is the Tsukuba FRP Bridge (photo credit: Iwao Sasaki), which is a pedestrian cable-stayed bridge completed in 1996 and located in Tsukuba, Japan; all stay cables of this bridge are made of CFRP. No. 2 is the Stork Bridge completed in 1996 and located in Winterthur, Switzerland (photo credit: EMPA); it is a highway cable-stayed bridge and two of its stay cables are made of CFRP. No. 3 is the Neigles CFRP Footbridge (photo credit: Tokyo Rope); it is a pedestrian suspension bridge completed in 1998 and located in Fribourg, Switzerland and its two main cables are made of CFRP. No. 4 is the Herning CFRP Bridge completed in 1999 and located in Herning, Denmark (photo credit: COWI), which is a pedestrian cable-stayed bridge with all cables made of CFRP. No. 5 is the Laroin CFRP Footbridge (photo credit: Freyssinet), which is a pedestrian cable-stayed bridge with 16 CFRP stay cables and 2 steel back stays; it was completed in 2002 and is located in Laroin, France. No. 6 is the Jiangsu University CFRP Footbridge completed in 2005 and located in Zhenjiang, China (photo credit: Kuihua Mei), which is a pedestrian cable-stayed bridge with all cables made of CFRP. No. 7 is the Penobscot Narrows Bridge, which is a highway cable-stayed bridge completed in 2006 and located in Penobscot, Maine, USA (photo credit: MOT); in 2007, six steel strands in the stay cables of this bridge were replaced by CFRP strands. No. 8 is the EMPA Bowstring Arch Footbridge completed in 2007 and located in Dübendorf, Switzerland (photo credit: Urs Meier); all the bowstrings of this bridge are made of CFRP. No. 9 is the TU Berlin CFRP Stress-Ribbon Footbridge completed in 2007 and located in Berlin (photo credit: Achim Bleicher); all stress-ribbons in this bridge are made of CFRP. No. 10 is the Cuenca Stress-Ribbon Footbridge completed in 2011 and located in Cuenca, Spain (photo credit: Mike Schlaich); it is a pedestrian stress-ribbon bridge with all stress-ribbons made of CFRP.

Future Trend of CFRP Cable Structures

As seen from the above section, the existing CFRP cable structures are all cable bridges. Moreover, majority of them are cable-stayed bridges. Up to now, there is not yet any CFRP cable roof or facade in the world. Furthermore, studies on CFRP cable roofs or facades are also very few (Feng et al. 2007) (Liu et al. 2015).

However, cable roofs and facades are ideal structures for CFRP cables, because most of them are orthogonally loaded by external loads. In orthogonally loaded cable structures, the structural stiffness is mainly comprised of the geometric stiffness, which is induced by the pre-tension force in the cables. The elastic stiffness generated by the elastic modulus of cables is less important. For such cable structures, increasing the tensile strength of cables to increase the pre-tension force is a more efficient way to raise the structural stiffness than increasing the elastic modulus of cables. Using CFRP cables, whose tensile strength is considerably greater than that of steel cables (Schlaich et al. 2015), in orthogonally loaded cable structures can improve the economic efficiency, although the elastic modulus of CFRP cables is smaller than that of steel cables and their unit price is also much higher (Schlaich
et al. 2015). Consequently, using CFRP cables in cable roofs and cable facades should be a trend for future buildings.

To investigate the feasibility of applying CFRP cables to cable roofs and cable facades, a small CFRP spoked wheel cable roof was built at TU Berlin in 2013, which is shown in Figure 2 (Schlaich et al. 2014).

![Figure 2 CFRP spoked wheel cable roof prototype](image)

In this prototype, the radial cables are loop-shaped CFRP tension members and the tension ring is a closed octagon CFRP loop. The cross section of the radial cable and the tension ring is 30 mm × 1.2 mm, which can bear nearly 80 kN tension force. Both types of CFRP structural elements were manufactured at TU Berlin by hand through laminating a continuous carbon fibre tow coated with epoxy resin on a rotating form and using vacuum technique to harden. The material of the compression ring, pillars and nodes is aluminium. To pre-tension the CFRP cables, each node of the tension ring was separated into two semilunar parts, which are linked together by four bolts. By tightening the bolts and hence shortening the distance between these two parts, the CFRP cable system of the spoked wheel cable roof was pre-tensioned to a sufficient level and a very stiff system was achieved.

**THEORETICAL BASIS**

*“Pin-Loaded” Concept*

As noted earlier, the key problem facing the application of CFRP cables in cable structures is how to anchor them. The advantages of CFRP cables can only be valid if their mechanical properties (i.e. strength and modulus) are fully exploited (Schlaich et al. 2012).

The conventional CFRP cable anchorages can be classified into two types, i.e. clamp anchorage and bond anchorage (Liu 2015). The clamp anchorage, as its name suggests, is a type of anchorage relying on the clamp function, or more specifically, the friction force and/or the mechanical interlocking force, to anchor the CFRP
cables. The bond anchorages rely on the bond force to anchor the CFRP cables. Usually, the bond materials used are the resin-based mortar (e.g. epoxy resin) or the cement-based mortar (e.g. high-expansion cement). These two types of CFRP cable anchorages have already been used in some projects, such as 6 existing CFRP cable stayed bridges (Figure 1). However, these anchorages are usually large, complicated and expensive. This makes them unfavourable for some CFRP cable structures, especially for cable roofs and cable facades, which usually require relative small and cheap anchorages.

The pin-loaded anchorage is a relatively new anchorage system for CFRP cables, which is different from the clamp or bond anchorage, as shown in Figure 3 (Liu 2015).

As can be seen from the above figure, the main part of pin-loaded anchorage is only a pin or a thimble. CFRP cables are anchored through winding on the pin. With such “pin-loaded” design concept, the anchorage structure can be greatly simplified and the length of anchorage can be considerably shortened. Moreover, when the anchored CFRP cable is subjected to tension, the pressure will be generated between the CFRP cable and the pin, which will generate an extra friction force and help to anchor the CFRP cable. The pin-load anchorages are particularly well suited for anchoring CFRP cables in cable roofs and cable facades, where the structural spaces are usually too limited to place the clamp anchorages or the bond anchorages. Moreover, beams and columns with round cross sections in such cable structures can be pins themselves for the pin-loaded CFRP cable anchorages and thus leading to even smaller anchorage sizes and simpler anchorage structures.

“Continuous Winded Band” Concept

The “continuous winded band” is a novel form of using CFRP. This idea was inspired by the pin-loaded anchorage. However, the CFRP cable used in such system is the CFRP continuous band, which will be wound through all the intermediate nodes and only anchored at the both end nodes (see Figure 4) or which will form closed loop without any anchorages (see Figure 6). The greatest advantage of the CFRP continuous winded band system is that the number of anchorages is minimised and thus exploiting to the full favourable conditions of CFRP cables and avoid the unfavourable ones.

TYPICAL FORMS OF CFRP CONTINUOUS WINDED BAND SYSTEM

There are several possible forms of CFRP continuous winded band system. Two typical forms are introduced in this section. Apart from these two examples, the CFRP continuous winded band system is well suited for many other cable structures.

One Typical Form: Anchored at End Nodes
First, one typical form in which the CFRP continuous band is anchored at end nodes is illustrated in Figure 4, which is the cable system of the cable membrane roof for a swimming pool. The details of the CFRP continuous band used and the nodes are illustrated in Figure 5. (Liu 2015)

![Figure 4 Swimming pool cable roof with CFRP continuous winded band system](image)

As can be seen from the above figures, only one cable, i.e., a CFRP continuous band, is used in the structure. This makes the structure concise and aesthetically pleasing. Furthermore, the CFRP continuous band is wound through every intermediate node and only anchored at end nodes with pin-loaded anchorages.

**Another Typical Form: Closed Loop**

Another typical form in which the CFRP continuous band forms a closed loop without any anchorage is illustrated in Figure 6. The structure is a spoked wheel cable roof. The details of the CFRP continuous band used and the nodes are shown in Figure 7 (Liu 2015).

![Figure 6 Spoked wheel cable roof with CFRP Continuous Winded Band System](image)
As can be seen from the above figures, there are only two cables in the structure, i.e. a CFRP continuous band acting as the spoke cables and a CFRP inner ring. This CFRP continuous band forms a closed loop and is wound around all nodes without any anchorages.

**CONCLUSION AND PROSPECTATION**

This paper mainly presents the conceptual design of the CFRP continuous winded band system. Adopting such system, the number of the required anchorages will be significantly reduced or eliminated and thus making the best use of the advantages of CFRP cables and bypassing their disadvantages. Furthermore, cable structures with the CFRP continuous winded band system are very concise and aesthetic. Admittedly, except for the advantages, the CFRP continuous winded band system also has some disadvantages. For example, the failure of a single segment of the CFRP continuous band may result in the failure of the whole system; therefore, the safety factor for the CFRP continuous band might be set higher than that for ordinary cables.

In general, the CFRP continuous winded band system is an interesting and promising way of using CFRP in cable structures. More researches including the static and dynamic properties and the construction methods should be done for its realisation.

**REFERENCES**


NEW FRONTIERS FOR THE USE OF FRP REINFORCEMENT IN GEOMETRICALLY COMPLEX CONCRETE STRUCTURES

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ABSTRACT

The development of flexible formwork has made it possible to cast optimised geometrically complex and thin walled reinforced concrete structures. At the same time, advanced composite materials offer the opportunity to solve the problem of steel corrosion, which can affect aging of concrete structures. With the goal of achieving sustainable design, being able to combine optimised geometries with durable construction materials is a major challenge for civil engineering.

New research at the University of Bath and the University of Miami aims to completely replace internal steel reinforcement in geometrically complex concrete structures with durable and ready-to-use cages made of fibre reinforced polymer (FRP) reinforcement. By fabricating the reinforcement in the desired geometry, it will be possible to provide the required strength exactly where needed, thereby reducing the amount of concrete required to resist internal forces and capitalizing on the extraordinary possibilities offered by both concrete and FRP construction materials.

The design of such optimized elements and the automated process of manufacturing the Wound FRP (W-FRP) reinforcement are presented in this paper.

KEYWORDS

W-FRP, RC beams, optimization, fabric formwork.

INTRODUCTION

Since its early age, concrete has been cast in rigid formwork, normally in the form of rectangular prismatic solids. Recent research has shown that nearly 40% of concrete that is currently used in building structures has little contribution to structural capacity, only adding extra self-weight to the building (Orr, Darby et al. 2011). Flexible formworks rely on the use of a system of flexible sheets of fabric to allow complex shapes to be easily cast, thus facilitating the construction of optimised concrete structures (Orr, Darby et al. 2011, Veenendaal, Coenders et al. 2011). This is crucial for the sustainable design of concrete structure, provided that the saving in material use directly lead to saving in embodied energy.

In addition to the great possibilities of building unconventional concrete geometries and reduce the material consumption, the use of fabric formwork offers a technological advantage. By casting concrete into a permeable fabric membrane, excess pore water is allowed to bleed from the concrete during curing. The resulting reductions of water/cement ratio in the surface zone of concrete elements bring improvements in durability (Price 1999, Orr, Darby et al. 2013).

Nevertheless, the need to assemble quite complex reinforcing cages together with the low durability of steel reinforcement in thin walled structures, are some of the reason why a large-scale deployment of this technology has not occurred yet.

The use of FRP as internal reinforcement can help to overcome these kind of issue. The substantial disadvantage inherent in the use of FRP reinforcement is related to onsite flexibility. Whereas steel reinforcement can easily be bent into an extensive range of geometries at the construction site, FRP reinforcement needs to be carefully shaped during manufacture according to the final demand. This challenge turns in favour of the pursuit of material
optimization, which require an increase in the level of control during the design and construction phases to produce complex geometries. Moreover, the terrific lightness of composite reinforcement suggests the opportunity to deliver prefabricated reinforcement cages to site, ready to be positioned and cast.

With all this in mind, coupling the geometric flexibility given by novel formworks systems with the enhanced durability offered by FRP reinforcement, gives the opportunity to depict a new paradigm of design for the reinforcement of concrete structures, aiming to pursue the sake of sustainability in the construction process. Further savings in emissions can be achieved using low-carbon cements, which further reduce the embodied energy of concrete. In fact, such cements cannot readily be used with steel reinforcement as they also reduce the alkalinity of the concrete mix. By contrast, a reduced alkalinity environment makes no difference when using a non-corrosive reinforcement, allowing the potential to achieve embodied energy savings through both geometry and material choices.

The present research aims to explore the possibility of fabricating FRP reinforcement in complex geometries by mean of an automated manufacturing system, in order to provide tensile strength exactly where it is needed by optimized RC structural elements. This will be transformative for concrete construction, as it will greatly simplify the design of more efficient, thin walled and architecturally daring concrete structures.

DESIGN PROCEDURE

Literature Review

FRP has excellent tensile properties in the direction of the fibres. Unlike steel, which exhibits yielding and plastic flow, it has a linear elastic stress–strain relationship. The Young’s modulus of FRP reinforcement is usually lower than steel as it is primarily dependent on the stiffness of fibre used (Nanni 1993). For this reason the serviceability limit states, rather than the ultimate limit states, are normally governing the design of FRP reinforced structures. In particular, the control of deflection is very often the most decisive check in the design process (Ascione, Mancusi et al. 2010, Nanni, De Luca et al. 2014). Additionally, the lack of yielding of the FRP reinforcement suggests the design of over reinforced sections, in order to obtain concrete crushing flexural failure and prevent sudden FRP rupture. This circumstance produces, in any case, an overall less ductile behaviour of FRP RC members as compared to steel RC members.

Another relevant design problem is modelling the mechanisms of shear resistance in FRP reinforced concrete members. Shear failure of reinforced concrete structures is brittle and it can be tremendously sudden and dangerous when dealing with FRP shear reinforcement (Matta, El-Sayed et al. 2013, Ascione, Razaqpur et al. 2014, Razaqpur and Spadea 2015). Furthermore, the most up-to-date design codes do not provide specific guidance to analyze the shear strength of non-prismatic concrete members (Orr, Ibell et al. 2014).

In order to develop a model that can efficiently predict the structural behaviour of such structural elements and consequently perform the structural optimization of fabric–formed concrete beams reinforced with FRP, a method of analysis with broad applicability was developed into a Matlab code. The geometry of the fabric formed member and the distribution of the reinforcement was modelled in a closed form allowing for considering variation in section dimensions along the members. The optimization criteria aim to obtain the minimal mass of concrete and observing the capacity design requirements as per the mostly recognized design codes (CSA S806 2012, CSA S6 2014, ACI 440.1R 2015).

Computational Method

The computational procedure followed can be briefly outlined as below:
1. Setting static scheme, structural materials, geometric limitations, design standard, capacity design requirements.
2. Finding the applied bending moments and shear stresses from the given loading.
3. Optimization for strength is carried out to find the profile of the beam which satisfy the ultimate limit states for flexure and maximize concrete contribution to shear strength. This is done dividing the section geometry and layout of the longitudinal reinforcement for each section.

At this point, a FRP RC fabric formed beam without shear reinforcement, optimized for flexure, is obtained.

4. The serviceability behaviour of the member is then assessed (elastic deflection, crack width and time dependent behaviour checks).
5. If the member fails to meet serviceability limit state conditions, the applied bending moments and shear stresses are virtually increased by a scaling factor, and the computational procedure go back to point 3 until point 5 is satisfied.

The reason for increasing the design stresses rather than performing a specific optimization process for serviceability is related to the willingness of observing the capacity design rules. At this point, a FRP RC fabric formed beam without shear reinforcement, optimized for flexure, and meeting the serviceability limit state conditions is obtained.

6. Optimization for shear strength is carried out to find the best geometry and the minimal quantity of web reinforcement which satisfy the ultimate states for shear. The web reinforcement geometry is limited to shapes that can be obtained by fibre winding, hence they may result in being a certain type of spirals. Also in this case the method is based on a sectional analysis.

At this point, a FRP RC fabric formed beam, optimized for flexure and shear strength, and meeting the serviceability limit state conditions, is obtained. The code is able to produce a STereoLithography (.stl output file) of the designed beam, which is suitable to embed all the information pertaining the geometry of the designed structural elements.

MANUFACTURING METHOD

As described in the following paragraphs, a computer controlled winding of impregnated carbon fibres around a set of FRP bars gave us the opportunity to obtain durable, lightweight, and ready-to-use reinforcement cages.

The manufacturing of web reinforcement is operated by means of a process based on the filament winding fabrication technique, which consist on wrapping continuous fibres under tension over a rotating mandrel. While the mandrel rotates, a wind eye on a carriage moves horizontally, laying down the fibres in the desired pattern. In the wet winding method, the fibre picks up resin either by passing through a resin bath or from a metered application system. In the dry winding method, the reinforcement is in the pre-impregnated form. After several layers are wound, the component is cured and removed from the mandrel. This method of manufacturing provides a great control over fibre placement and uniformity of the material structure and it is generally used to produce continuous hollow shapes with constant cross section.

In the present application, slightly curved CFRP bars, responsible for providing the flexural strength to the concrete beams, are attached to the mandrel according to the prefigured reinforcement geometry. A refined system of control allows the winding of a number of carbon tow layers in the form of spirals with variable cross section. After the winding and curing process occurs, the reinforcing bars are maintained in the curved configuration by the wound reinforcement.

The filament winding prototype, specifically designed and constructed at the University of Bath with the aim of producing this new class of reinforcing cages, is shown in Figure 1. The design includes the possibility of implementing either wet-winding, by means of a resin bath, or a dry-winding method.

In this work a continuous 50k carbon fibre tow (C T50-4.0/240-E100) produced by Sigrafil (SGL group) is adopted. The SGL Carbon tow is used in combination with Fyfe Tyfo S two-component epoxy to implement a wet-winding process. This class of epoxy resin, suitable for the wet-layup of external strengthening of structural members, can be applied at room temperature and air cured. Both are considerable advantages for this application as the resin bath does not need to be heated and the curing process can be simply operated by storing the cages at standard lab conditions for 48h-72h.

FLEXIBLY FORMED BEAMS

With the aim of validating both the optimization procedure and the W-FRP reinforcement cages manufacturing method, six FRP fabric formed beams were designed and cast. The beams are currently being tested at the University of Bath.
Assumptions

The adopted static scheme is a simply supported beam with three meters span and half-meter overhang on each side. The entire beam is subject to a uniformly distributed load.

Two different study cases were taken in consideration:

1) The first set of beams (Set I) is intended to simulate a precast fabric formed joist supporting a lightweight floor (e.g. all-FRP or wood floor).
2) The second set of beams (Set II) aims to reproduce the use of a precast fabric formed beam with an in-situ casting of a concrete floor. In the experimental work described below, the beam and the slab elements were cast together for ease of construction.

Additionally, the following assumptions were made for design purposes:
- The dead load and the live load are 2.5 kN/m and 7.5 kN/m, respectively;
- The strength class of concrete is C30/37;
- The bottom and top reinforcement are respectively #3 Carbon and #3 Glass FRP Aslan bars produced by Hughes Brothers.

Optimization Results

Table 1 shows a comparison of the most relevant details of the beams. A 3D visualization of the beam’s Stereolithography, as generated by the design code, is also shown in Figure 2.

Each set is composed of three beams having identical concrete geometry and longitudinal reinforcement but different W-FRP shear reinforcement. Whereas they all satisfy the points 1 to 5 of the above-mentioned computational procedure, only beams x.3 have the requested shear strength (point 6). In fact, beams x.1 and x.2 are purposely designed to exhibit a premature and sudden shear failure.

Table 1 – Details of the beams

<table>
<thead>
<tr>
<th></th>
<th>Set I</th>
<th>Set II</th>
</tr>
</thead>
<tbody>
<tr>
<td>beam length</td>
<td>4100 mm</td>
<td>4100 mm</td>
</tr>
<tr>
<td>flange width</td>
<td>300 mm</td>
<td>900 mm</td>
</tr>
<tr>
<td>flange thickness</td>
<td>60 mm</td>
<td>60 mm</td>
</tr>
<tr>
<td>web minimum width</td>
<td>85 mm</td>
<td>85 mm</td>
</tr>
<tr>
<td>beam depth at midsap</td>
<td>265 mm</td>
<td>190 mm</td>
</tr>
<tr>
<td>beam depth at supports</td>
<td>180 mm</td>
<td>150 mm</td>
</tr>
<tr>
<td>beam depth at ends</td>
<td>95 mm</td>
<td>110 mm</td>
</tr>
<tr>
<td>top reinforcement at supports</td>
<td>2 x #3 GFRP</td>
<td>3 x #3 GFRP</td>
</tr>
<tr>
<td>bottom reinforcement at midsap</td>
<td>3 x #3 CFRP</td>
<td>4 x #3 CFRP</td>
</tr>
<tr>
<td>Concrete volume</td>
<td>0.14 m$^3$</td>
<td>0.27 m$^3$</td>
</tr>
<tr>
<td>number of wound CFRP layers (x.1 / x.2 / x.3)</td>
<td>0 / 3 / 5</td>
<td>0 / 3 / 5</td>
</tr>
</tbody>
</table>

In detail:
1) Beams I.1 and II.1, having no shear reinforcement, are expected to fail in shear, due to shear tension failure;
2) Beams I.2 and II.2, having three layers of wound reinforcement, are also expected to fail in shear, due to shear tension failure and showing wound reinforcement rupture.

3) Beams I.3 and II.3, having 5 layers of wound reinforcement, are expected to fail in flexure, due to concrete crushing at midspan.

![3D visualization of the beam’s Stereolithography](image)

**Construction**

The essence of flexible construction is to secure the fabric on a supporting frame in order to achieve the desired form once the formwork is filled with concrete. In the present work, the fabric is draped into a plywood supporting frame to shape the non planar lateral surface of the stems whereas the control over the beam elevation is achieved using a keel, pre-cut to the desired elevation (Figure 3a).

Figure 3b shows the W-FRP cage of beam II.2 instrumented with strain gauges and installed into the fabric formworks before concrete casting.

![Formworks cross-section diagram](image)

![Instrumented W-FRP cages](image)

**Figure 3 a) formworks cross-section diagram; b) instrumented W-FRP cages installed into the fabric formworks before concrete casting (beam II.2).**
EXPERIMENTAL INVESTIGATION

The experimental setup (Figure 4) consists of two simple supports and seven hydraulic jacks attached to a rigid frame, each one instrumented with a load cell. Five jacks are equally spaced on the 3000 mm beam span and powered by the same oil circuit, as to apply a load $P$. The remaining three jacks are installed at the ends of the cantilevers (500 mm long), and powered by a separate different hydraulic circuit in order to apply a load equal to $0.5 \cdot P$. This loading scheme allows to simulate a uniformly distributed load.

Seven displacement transducers are installed: one at the beam midspan, two at the supports, two at the quarters span and two at the ends of the cantilevers. Five cross-sections of the beams are instrumented with uniaxial strain gauges installed on the reinforcing bars and on the concrete (cyan sections in Figure 4) in order to monitor the flexural curvature of the beams. The wound reinforcing cages of beams x.2 and x.3 are instrumented with uniaxial strain gauges mounted on each leg included between the supports and the closer point load in the span (magenta areas in Figure 4). The front face of each beams is painted with a dotted bi-chromatic pattern. High resolution pictures, perpendicular to the observed surface, are taken at each 2kN increment throughout the entire loading cycle to enable subsequent analysis using Digital Image Correlation (DIC).

REMARKS AND FUTURE WORK

This paper presents an analytical tool able to show the potential of using bespoke CFRP reinforcement in RC optimised beam and the automated process of manufacturing the FRP reinforcing cages. Such structures have complex geometries, which are difficult to reinforce with conventional steel. The technical developments in this paper provide the basis for a novel alternative reinforcement technique that is well suited to automation and mass production.

The experimental validation is under development; hence some of the results will be presented during the CICE 2016 conference, in Hong Kong.

ACKNOWLEDGMENTS

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DATA ACCESS STATEMENT

All data created during this research are openly available from the University of Bath data archive at http://doi.org/10.15125/BATH-00213

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PERFORMANCE AND STRENGTH OF PULTRUDED GFRP ANGLE STRUTS
SUBJECT TO CONCENTRIC COMPRESSION

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ABSTRACT

This paper aims to investigate performance and strength of glass-fiber reinforced polymer (GFRP) pultruded equal leg angles subject to short-term concentric compression. The background theory for a perfect angle strut is presented and the critical buckling stresses for two different boundary conditions are compared with those obtained with Generalized Beam Theory (GBT), showing good agreement. It is shown that, for typical pultruded GFRP with relatively low shear modulus, the influence of the flexure about major axis, shear and local deformations are negligible. The real column behavior is discussed and a column strength equation combining beam-column and classical plate theories is proposed to address the problem. Finally, the experimental results available in literature are compared with the lower-bound strength curve, leading to the conclusion that the method is adequate to predict ultimate load of angle struts, although more tests are needed for calibration of imperfection parameters.

KEYWORDS

GFRP, angle, column strength curve, flexural-torsional buckling, flexural buckling.

INTRODUCTION

Extensive research has been conducted worldwide on the behavior of pultruded columns and different strength curves have been proposed in literature (Barbero and Tomblin, 1994, Barbero and DeVivo, 1999, Seagatith and Sriboonlu, 1999, Puente et al., 2006, Cardoso et al., 2014a). However, major effort has been directed toward the performance of doubly symmetric sections, where an interaction between local and flexural global buckling modes governs the behavior. To date, few works have been focused on the compressive behavior of monosymmetric or asymmetric cross-sections and no provisions for these sections have been included in the current Italian code for design of pultruded members (CNR, 2008) and in the Prospect for the New Guidance in the Design of FRP (Ascione, 2016). The work of Zureick and Steffen (2000) remains the only dedicated to investigate the performance of pultruded angle struts, despite their intensive use as truss members (e.g. footbridges and braced frames). In that work, the authors tested 25 equal-leg angle columns having different width-to-thickness (b/t) ratios and lengths and recommended the nominal strength to be determined as the lesser of material compressive strength, flexural buckling and flexural-torsional buckling critical stresses. Additionally, the authors proposed design factors computed from reliability analysis of their experimental results. Their findings have been included in the draft for the future ASCE Pultruded Fiber Reinforced Polymer (ASCE, 2010). Although simple, this method does not represent the actual behavior, where imperfections are present and second-order stresses have to be considered. A recent approach proposed by Dinis et al. (2015) for design of steel thin-walled angle columns, for instance, take into account imperfections and interaction between buckling modes. This paper aims to complement Zureick and Steffen’s (2000) work by describing the real column behavior and proposing a strength curve that can be used to predict a lower bound member capacity.

PERFECT COLUMN

The term ‘perfect column’ adopted in this work refers to perfectly straight and undistorted members subject to a concentric compression force. For a perfect angle strut, three distinct and uncoupled failure modes may occur: flexural-torsional buckling, flexural buckling and material compressive failure. The two first modes are associated with instability limit state, while the latter occurs if the column material compressive strength is exceeded prior to buckling.
Crushing

The term ‘crushing’ generally refers to all collapse modes that may occur at the constituent material level of a fibrous composite material: elastic microbuckling, plastic microbuckling and fiber crushing (Fleck, 1997). Comprehensive closed-form equations for GFRP material crushing strength ($F_{cr}$) have been presented by Barbero (1998) and Barbero et al. (1999), but input data for the method are rarely presented in data sheets, tables and manuals provided by manufacturers. Recently, Cardoso et al. (2014b) proposed an empirical expression based on linear regression of mean experimental results obtained by various authors. Design codes usually specify minimum material compressive strength for structural application.

Flexural Buckling

The flexural buckling ($F$) about the minor axis may occur in relatively long struts and the critical stress, $F_{cr,Fy}$, is given according to the so-called Euler buckling equation:

$$F_{cr,Fy} = \frac{\pi^2 E_t}{(K, L/r_t)}$$

in which $E_t$ is the cross-section representative modulus of elasticity; $L$ is the column length; $K$ is the flexural buckling coefficient about the minor axis; and $r_t$ is the radius of gyration. It is important to mention that this expression does not consider shear deformation influence.

Flexural-Torsional Buckling

Flexural-torsional buckling ($FT$) is mainly characterized by a natural coupling of flexural buckling about major axis and torsion about shear center. This mode is likely to occur for thin-walled open cross-sections with low torsional rigidity and with shear center not coinciding with the centroid, e.g. angles and channels. The flexural-torsional buckling critical stress, $F_{cr,FT}$, for a monosymmetric section is given as (Timoshenko and Gere, 1961):

$$F_{cr,FT} = \frac{F_{cr,Fy} + F_{cr,T}}{2}\left[1 - \frac{4F_{cr,Fy}F_{cr,T}[(1-(x_0/L_t)^2)]}{(F_{cr,Fy} + F_{cr,T})^2}\right]$$

in which $x_0$ is the distance between the shear center and the centroid; $L_t$ is the cross-section polar radius of gyration with respect to the shear center; $F_{cr,Fy}$ is the flexural buckling stress about major axis; and $F_{cr,T}$ is the torsional buckling critical stress. For equal-leg angles with negligible transverse shear deformation, $F_{cr,Fy}$ and $F_{cr,T}$ can be determined according to Eqs. 3 and 4, respectively (Zureick and Steffen, 2000):

$$F_{cr,Fy} = \frac{\pi^2 E_t}{(K, L/r_t)}$$

$$F_{cr,T} = \frac{\pi^2 E_t}{12(1-\nu_{LT}\nu_{TT})}\left(\frac{t}{K, L}\right)^2 + G_{LT}\left(\frac{t}{b_t}\right)^2$$

where $K$ is the flexural buckling coefficient about the major axis; $K$ is the torsional buckling coefficient, respectively; $r_t$ is the radius of gyration about the major axis; $b_t$ and $t$ are angle leg width and thickness, respectively; $E_t$ is the wall flexural modulus in longitudinal direction; $G_{LT}$ is shear modulus; and $\nu_{LT}$ and $\nu_{TT}$ are the major and minor in-plane Poisson’s ratio, respectively. It is important to mention that $E_{LT}$ is highly dependent on the roving layers distribution throughout wall and may be considerably lower than $E_t$ for thinner sections. For thicker sections, the difference becomes negligible (Cardoso et al., 2014b).

For typical pultruded GFRP material, shear modulus is very low in order that torsional buckling ($T$) governs the behavior. Zureick and Steffen (2000) assert that $F_{cr,FT} \sim 0.9F_{cr,T}$. However, it can be shown that the simply assumption of $F_{cr,Fy} \sim F_{cr,T}$ is sufficiently accurate for most of the cases. This hypothesis – $FT \sim T$ – is assumed in this work and the differences are discussed in the following section.

Figure 1 presents a typical cross-section with nomenclature and reference axes adopted in this work. The cross-section deformations for the buckling modes discussed previously are also illustrated.
Signature Curve

Although the dominant modes are flexural-torsional and flexural, angle sections may also exhibit local bending and shear deformation (Dinis et al., 2010). To evaluate the accuracy of Eqs. 1 and 4 \( F_{cr,FT} \sim F_{cr,T} \), a comparison was made with the results obtained with the Generalized Beam Theory (GBT) (Schardt, 1994), implemented using the software GBTul (Bebiano et al., 2008), for two usual end conditions: pinned-ends with simply supported walls (PP-SS), for which \( K_x=K_y=K_t=1.0 \); and fixed-ends with clamped walls (FF-CC), where \( K_x=K_y=K_t=0.5 \). In Figure 2, critical buckling stress is plotted against the column length – the so-called signature curve – using Eqs. 1 and 4 for a 127x9.5 mm angle section and typical material properties. Values obtained with GBTul including global, local and shear deformation modes are also plotted for certain lengths. A maximum localized difference of 8% was observed, which indicates that the equations adopted are sufficient accurate for a design approach. Other important conclusions can be drawn from GBTul analyzes: i) modes associated with wall transverse bending do not play important role and their influence may be neglected; and ii) shear deformation modal participation was about 18% and 14% for short pinned and fixed-ended columns, respectively, but this seems not to influence importantly the critical stress. A similar difference can be observed for other cross-sections.

**Signature Curve** \( F_{cr} \times L \)

- \( b_x = 122.3 \) mm
- \( t = 9.5 \) mm
- \( E_{L,f} = 21,000 \) MPa
- \( E_L = 21,000 \) MPa
- \( G_{LT} = 4000 \) MPa
- \( (1-\nu_{LT}\nu_{TL}) = 0.95 \)

Figure 2 Critical stress versus length (signature curve) for an angle strut 127x9.5 mm (FT = flexural-torsional; T = torsional; F = flexural).

REAL COLUMN

Real columns are those affected by geometric and material imperfections. Applied compressive force produces second-order bending moments that result in increased lateral deflections and, therefore, reduced critical capacities compared to those described as perfect columns (Batista, 2004). A similar behavior is expected for the column constituent plates, which experience second-order effects associated with wall imperfections.
To derive a strength equation, it is initially considered the only influence of the flexural-torsional mode. Assuming that this mode can be approximately described by a cross-section rotation about the shear center ($F_{c,FT} - F_{c,T}$), the constituent plates can be isolated and analyzed with a classical plate theory. The following hypotheses are considered: i) stretching in the middle plane and post-buckling behavior are neglected; ii) material behavior is linear elastic; iii) mechanical properties and thickness are uniform throughout the plate; and iv) residual stresses are negligible. An analytical model similar to the one presented by Cardoso et al. (2014a) can be developed and the following normalized flexural-torsional strength ($\chi_{FT}$) equation can be obtained:

$$\chi_{FT} = \frac{F_x}{F_{pp}} = \frac{1 + \alpha_{FT} + \lambda_{FT}^2 - \sqrt{(1 + \alpha_{FT} + \lambda_{FT}^2)^2 - 4\lambda_{FT}^2}}{2\lambda_{FT}^2}$$

(5)

where $F_{o,FT}$ is the ultimate load associated with the flexural-torsional mode; $\lambda_{FT} = (F_{Lc}/F_{c,FT})^{1/2} - (F_{Lc}/F_{o,FT})^{1/2}$ is the FT-mode relative slenderness, and $\alpha_{FT}$ is the FT imperfection factor. Although $\alpha_{FT}$ can be written in terms of the wall geometry, initial imperfection, plate stiffness and strength parameters, this factor can also be determined experimentally or empirically derived to cover the experimental results (Cardoso et al., 2014a).

It is interesting to note that Eq. 5 has the same form as the typical column strength curves developed for steel, such as those defined by Perry-Robertson (Ziemian, 2010) and Dutheil (1961). As expected, for perfect plates ($\alpha_{FT} = 0$) the strength reduces to $F_{Lc}$ for $\lambda_{FT} \leq 1.0$ and to $F_{c,FT} - F_{c,T}$ for $\lambda_{FT} > 1.0$.

To consider the influence of the flexural buckling mode, the following hypotheses are assumed: i) column bending stiffness remains constant until failure, i.e., bending stiffness is not affected by flexural-torsional buckling; and ii) failure occurs when the compressive stress reaches its capacity, taking into account the reduction associated with flexural-torsional. An analytical model based on the beam-column theory similar to the one presented by Cardoso et al. (2014a) can be developed and the following final normalized strength equation can be obtained:

$$\chi = \frac{F_x}{F_{pp}} = \frac{1 + \alpha_{F} + \lambda_{F}^2 - \sqrt{(1 + \alpha_{F} + \lambda_{F}^2)^2 - 4\lambda_{F}^2}}{2\lambda_{F}^2}$$

(6)

in which $\chi$ is the normalized strength; $F_x$ is the ultimate load taking into account combination of flexural (F) and FT modes; $F_{pp}$ is the ‘perfect plate’ strength given as the lesser of $F_{Lc}$ and $F_{c,FT}$; $\alpha_{F}$ is the column imperfection factor; $\lambda_{F} = (F_{pp}/F_{c,F})^{1/2}$ is the F-mode relative slenderness; $\rho_{FT} = \chi_{FT}/\chi_{FT,0}$ represents the reduced capacity with respect to the perfect condition for the FT-mode; and $\chi_{FT,0}$ is the perfect plate normalized strength, i.e., $\chi_{FT,0} = 1.0$ for $\lambda_{FT} \leq 1.0$ and $\chi_{FT,0} = 1/\lambda_{F}^2$ for $\lambda_{FT} > 1.0$. $\alpha_{F}$ can be written in terms of the column geometry and initial imperfections or determined experimentally to cover the experimental results (Cardoso et al., 2014a).

Once more, the equation derived is similar to typical column strength curves. It is important to note that, as expected, for perfectly straight columns ($\alpha_{FT} = 0$) the strength reduces to $\rho_{FT}F_{pp}$ for $\lambda_{F} \leq 1.0$ and to $F_{c,X}$ for $\lambda_{F} > 1.0$. If no imperfections are present (perfect column condition), $\rho_{FT} = 1.0$ and the ultimate load reduces to the lesser of $F_{o,FT}, F_{o,c,F}$ and $F_{Lc}$. The proposed expression allows determination of angle-strut ultimate load based on flexural-torsional and flexural slendernesses and imperfection factors.

**COMPARISON WITH EXPERIMENTAL RESULTS**

In this section, the experimental results reported by Zureick and Steffen (2000) are used to evaluate the method. Computation of relative slenderness, $\lambda_{FT}$, for all struts tested by these authors resulted in values greater than 2.0, indicating that all angles tested were slender with respect to flexural-torsional buckling mode. For an arbitrarily assumed imperfection factor $\alpha_{FT} = 0.1$, it can be seen that reduced capacity factors, $\rho_{FT}$, are similar for all columns (average $\rho_{FT} = 0.98$). In Figure 3, a normalized strength curve with $\alpha_{FT} = 0.45$ is proposed and it can be seen that all test results but one are adequately covered.

To understand the influence of flexural-torsional and flexural slendernesses in the member capacity, a contour plot of the reduced strength factor with respect to the perfect column condition, $\rho = F_x/\min(F_{Lc}, F_{c,FT}, F_{o,FT})$, as a function of $\lambda_{F}$ and $\lambda_{FT}$ is presented in Figure 4. Zureick and Steffen’s (2000) results are also shown in the ‘map’ with their corresponding $\rho_{exp}$. Two important conclusions can be drawn from this plot: i) the maximum reduction of capacity is expected for columns having $\lambda_{F} = \lambda_{FT} = 1.0$; and ii) there is a lack of experimental data in this critical region, indicating that more tests are needed.

**CONCLUSIONS**

This paper presented a brief review on the behavior of perfect with orthotropic material. Critical buckling equations were compared with the results obtained from a GBT analysis, showing that flexure about major axis, wall
Transverse bending and shear deformations do not affect significantly the critical loads for typical pultruded material and their influence can be neglected.

The concepts associated with the behavior of a real angle strut were also presented and a column strength equation derived from classical plate and beam-column theories was proposed (Eq. 6). The approach is simple and considers the influence of both flexural-torsional and flexural slendernesses and the imperfection factors associated with each mode. Although these imperfection factors can be explicitly written in terms of imperfections, geometric and material properties, it seems convenient to obtain them experimentally.

The method was validated using experimental results available in literature. In this study, arbitrary imperfection factors were used in the evaluation. Finally, a contour plot of the reduced capacity with respect to the perfect condition is presented, showing that maximum reduction occurs for columns having $\lambda_F = \lambda_{FT} = 1.0$. Moreover, more tests with different slenderness combination are needed to augment the available data and to obtain reliable imperfection factors. An experimental program is currently underway at Pontifical Catholic University of Rio de Janeiro with this purpose.

Figure 3 Normalized strength versus column slenderness associated with flexural buckling mode (column strength curve).

Figure 4 Contour plot of the reduced strength factor as a function of both flexural and flexural torsional slendernesses.
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ABSTRACT

The rotational behavior of the web-flange junctions (WFJs) of a pultruded GFRP bridge deck system was investigated. The rotational response of three WFJ types was characterized. An experimental procedure based on three-point bending and cantilever experiments conducted on the web elements and simple analytical models was used. The WFJs generally exhibited non-rigid and nonlinear behavior. The overall moment-rotation relationships, rotational stiffness, strength and failure modes differed depending on the web type, the location of the WFJ within the deck profile, the initial imperfections and the direction of the bending moment applied. Simplified expressions to model the WFJ rotational behavior were derived. The validity of the experimental and idealized rotational responses was assessed by means of numerical simulations of full-scale experiments conducted on the GFRP deck.

KEYWORDS

GFRP, pultrusion, bridge deck, web-flange junction, rotational stiffness, mechanical testing.

INTRODUCTION

Pultruded glass fiber-reinforced polymer (GFRP) bridge decks are one of the most developed and extended applications of FRP materials for load-bearing structural members in the civil engineering domain. GFRP bridge decks fulfill two structural functions: (i) distribution and transmission of the traffic loads applied to the bridge to the underlying superstructure; (ii) participation in load transfer in the bridge’s longitudinal direction by acting as the top chord of the main girders when there is sufficient composite action between the girder and deck. For pultruded GFRP deck systems, exhibiting orthotropic structural behavior, the performance concerning the two aforementioned functions is influenced by the deck’s behavior in its transverse-to-pultrusion direction.

In a previous study (Yanes-Armas et al. 2016a), the transverse behavior of a GFRP deck with trapezoidal cell geometry, DS (DARPA 2000), was experimentally investigated. The slab cross section, the loading configuration and the load-deflection response are illustrated in Figure 1(a) and (c). The load transfer mechanism, failure mode and system transverse in-plane shear stiffness were studied. The deck exhibited a frame-governed behavior whereby the load was mainly transmitted by local shear and bending moments in the web and flange elements. Progressive cracking occurring in the web-flange junctions (WFJs) resulted in a stiffness reduction without leading to the deck’s final failure. A non-brittle failure occurred and a sustained load-bearing capacity under the development of large displacements was recorded. A basis for evaluating the deck’s performance was established; additional local features, particularly the rotational behavior of the WFJs, required further evaluation.

The objective of this work was to characterize the rotational behavior of the DS WFJs. Three WFJ types, in two bending moment directions each, were investigated. A method based on three-point bending and cantilever experiments conducted on the web elements and simple analytical models was used. Simplified expressions to model the WFJ rotational behavior were derived. Lastly, their validity was assessed by means of numerical simulations of full-scale experiments conducted on the deck.

EXPERIMENTAL PROGRAM

Experimental Approach

Each DS WFJ may be defined by three convergent elements (one web and two flange segments). The typical fiber architecture of the WFJ area, consisting of E-glass fiber reinforcements embedded in an isophthalic polyester resin, is illustrated in Figure 2. Based on the fiber architecture, the flange can be considered as being continuous across
the WFJ. On the other hand, the web’s end may exhibit a certain rotational flexibility due to: (i) the lack of fiber continuity towards the flange; (ii) the change of direction of the prolonged fabrics; (iii) the roving core/resin-rich area, see Figure 2. The end condition of the web can thus be considered as a semi-rigid joint whose rotational behavior is described by the relationship between the local bending moment applied to the web’s end, $M$, and the relative rotation between the web and the flange elements, $\phi$. Hence, the web’s end can be modeled in the form of a rotational spring responding to the moment-rotation ($M-\phi$) relationship.

Figure 1 Experimental configuration for DS deck specimens subjected to transverse (a) three-point bending and (b) in-plane shear; measured and calculated load-displacement behavior under (c) three-point bending and (d) in-plane shear.

Figure 2 Fiber architecture and structural idealization of WFJs with (a) inclined and (b) vertical webs; dimensions in mm.

The complete $M-\phi$ relationship characterizing the rotational stiffness of the WFJs can be determined from three-point bending beam and cantilever experiments conducted on the web element. The rotational stiffness can be derived from the experimental responses obtained by using two simple analytical models: that of a simply supported beam under three-point bending and that of a cantilever with semi-rigid end condition, whose rotational stiffness equals that of the WFJ.

Figure 3(a) illustrates a simply supported beam of $L_{pb}$ span length subjected to symmetric three-point bending, which can be considered as two cantilevers of $L_{cant} = L_{pb}/2$ length with fixed end and symmetrically placed about mid-span, see Figure 3(b). Equal deflections under the load application point are obtained for both the three-point bending ($\delta_{pb}$) and the cantilever ($\delta_{cant}$) configurations given that $P_{cant} = R_{pb}$, i.e., $P_{cant} = P_{pb}/2$:

$$\delta_{cant,f}(P_{cant}) = \delta_{pb}(2P_{cant})$$

Figure 3(c) illustrates a cantilever beam with semi-rigid end condition, which is modeled as a rotational spring. The deflection of the partially fixed cantilever under the load application point, $\delta_{cant,ir}$, can be formulated as:

$$\delta_{cant,ir}(P_{cant}) = \delta_{cant,f}(P_{cant}) + \delta_e(P_{cant})$$
where $\delta_{cant,f}$ = deflection of the fixed end counterpart; $\delta_{\phi}$ = deflection caused by the rotation in the semi-rigid end. Operating with Eqs 1 and 2, $\delta_{\phi}$ can be expressed as:

$$\delta_{\phi}(P_{cant}) = \delta_{cant,sr}(P_{cant}) - \delta_{3pb}(2P_{cant})$$

(3)

Additionally, $\delta_{\phi}$ can be calculated as a function of the rotation in the semi-rigid end, $\phi$, as:

$$\delta_{\phi} = L_{cant}tg(\phi)$$

(4)

Therefore, by replacing Eq. 3 in Eq. 4, $\phi$ can be formulated as a function of the deflections, under the load application points, in the simply supported beam and the semi-rigid end cantilever configurations as:

$$\phi = arctg\left(\frac{\delta_{\phi}(P_{cant})}{L_{cant}}\right) = arctg\left(\frac{\delta_{cant,sr}(P_{cant}) - \delta_{3pb}(2P_{cant})}{L_{cant}}\right)$$

(5)

The bending moment acting in the semi-rigid end, $M$ (see Figure 3(c)), can be calculated as:

$$M = P_{cant}L_{cant}$$

(6)

In accordance with the preceding explanations, the $M-\phi$ relationship of the WFJ can be obtained from the recorded loads, measured deflections and set-up geometry of three-point bending beam and cantilever experiments performed on the web element, providing that $L_{cant} = L_{3pb}/2$. First, the web is subjected to three-point bending with simply supported end conditions and a $L_{3pb} = 130$-mm span length. Subsequently, the web-cantilever experiment is conducted with a $L_{cant} = 65$-mm lever arm – the WFJ is considered as a rotational spring and the experiment responds to the model illustrated in Figure 3(c). Finally, the WFJ $M-\phi$ relationship is calculated from the load-deflection data, by applying Eqs 5 and 6 throughout the whole $P_{cant}$-$\delta_{cant,sr}$ range and the corresponding $P_{3pb}$-$\delta_{3pb}$ $(2P_{cant}$-$\delta_{3pb})$ range.

Figure 3 Models of (a) simply supported beam under symmetric three-point bending; (b) cantilever beam with fixed end; (c) cantilever beam with semi-rigid end

**Specimens, Experimental Set-ups and Procedure**

Two groups of WFJs can be differentiated in the DS deck depending on the corresponding web geometry: vertical (V) and inclined (I). The latter can be classified into two types according to the closer (c) / farther (f) location of the adhesively-bonded profile-to-profile joint in the flange in relation to the web, see Figure 4(a).

The rotational stiffness of the three WFJ types (Ic, If, V) was investigated. The tensioned side of the web towards the obtuse/acute angle (I-WFJs) or towards the double / single flange laminate (V-WFJs) is denoted by o/a and d/s, respectively, see Figure 4(a). The experimental program was performed on three specimens from each of the six series. The specimens’ overall geometry (nominal width $b_o$ of 50 mm) is shown in Figure 4(a); their location within the deck panel of the preceding beam experiments (Yanes-Armas et al. 2016a) is shown in Figure 4(b). The visual inspection of the specimens revealed that partially bonded areas were apparent in the adhesive layer of the V-WFJs and that a small crack (referred to as “pre-crack” in the following) existed in the flange of every Ic specimen, see Figure 2(a). Differences in the actual fiber arrangement of the If- and Ic-WFJs were observed, despite their identical fiber architecture design; these are detailed in Yanes-Armas et al. (2016b).

Figure 4 (a) WFJ specimens; (b) location of WFJ specimens within DS deck panel when subjected to transverse bending (local bending moments in webs are shown); dimensions in mm
First, the webs of the WFJ specimens were loaded in a symmetric three-point bending configuration with simply supported end conditions and a 130-mm span length ($L_{pb}$), see Figure 5(a). The rotations and the vertical deflections along the specimen’s web were measured by means of a video extensometer. All experiments were performed under displacement control at a rate of 0.01 mm/s. The specimens were loaded until the strain in the web reached a limit value of 0.6% and were subsequently unloaded at a 0.02-mm/s rate. Then, cantilever experiments with a 65-mm lever arm ($L_{cant}$) were performed. A single test rig was designed and manufactured so that the web-cantilever experiments could be conducted on the six WFJ series using a universal testing machine, see Figure 5(b). The specimen’s flange was laterally clamped on both sides of the WFJ; the 20.3-mm-thickness flange area close to the WFJ remained unclamped. A video extensometer was used to measure the rotations of the specimen’s flange and web and the vertical deflections along the length of the latter. All experiments were conducted under displacement control at a 0.01-mm/s rate up to failure.

![Figure 5 Set-up for (a) three-point bending and (b) cantilever experiments](image)

**EXPERIMENTAL RESULTS AND DISCUSSION**

The average $M$-$\phi$ curves obtained for all series by applying the aforesaid procedure are shown in Figure 6 (see $exp$ curves). The normalized $M$ values (moment per unit width, $M/b$) are displayed. The ultimate bending moment of the WFJ ($M_{ult}/b$), the corresponding rotation ($\phi_{M,ult}$) and the initial tangent rotational stiffness, $K_{0,ult}/b$, defined as the slope at the origin of the $M$-$\phi$ curve, were calculated. The If-$w$WFJs initially displayed perfectly rigid behavior ($K_{0,ult}/b = \infty; \phi = 0$), irrespective of the direction of the local bending moment applied to the web element, almost up to their maximum ultimate bending moment. Their rotation at failure was negligible. The maximum moment capacity of both series was similar regardless of the examined bending direction. Nevertheless, after the peak moment, a sudden moment decrease occurred in the If-$o$ specimens while the If-$a$ specimens showed a progressive softening branch. A nearly constant post-peak (residual) moment capacity followed in both series, which was approximately 10% higher in If-$a$ than in If-$o$, although it developed up to an around 40% smaller maximum rotation. On the other hand, the Ic-$w$WFJs initially exhibited semi-rigid ($0 < K_{0,ult}/b < \infty$), nonlinear behavior preceding the maximum moment capacity, which occurred at rotation values significantly larger than in the If-$w$WFJs. The rotational flexibility of the Ic-$w$WFJs was related to their lower stiffness and initial nonlinearity in the cantilever configuration, associated with the flange’s pre-crack (see Yanes-Armas et al. 2016b). The Ic-$o$ specimens, where the applied local bending moment produced tension in the WFJ’s obtuse angle and hence opening of the pre-crack, displayed a more pronounced nonlinearity and an approximately 30% lower rotational stiffness. The Ic-$a$ specimens exhibited an approximately 40% higher peak moment than the Ic-$w$ specimens and a relative rotation at failure 3.3 times greater. The V-WFJs showed a non-rigid, slightly nonlinear initial $M$-$\phi$ response with comparable rotational stiffness irrespective of the bending moment direction – a 17% difference in $K_{0,ult}/b$ was found between the V-$d$ and V-$s$ series. However, the V-$s$ specimens exhibited a more pronounced nonlinearity and significant stiffness reduction as from approximately 70-80% of their maximum moment. A roughly constant post-peak moment capacity was exhibited by both the V-$d$ and the V-$s$ specimens. The average value of the residual moment-bearing capacity was approximately 30% higher in the V-$d$ series than in the V-$s$ series, although it developed up to an about 20% smaller maximum rotation. Differences in the rotational stiffness and/or strength of the six WFJ specimen types indicated that their characterization should be considered separately and that prospective modeling should take the dissimilarities into account.
MODELING

Empirical Modeling of Rotational Stiffness of Web-Flange Juncions

Based on the average $M$-$\phi$ curves obtained from the three-point bending and cantilever experimental results (see exp curves in Figure 6), two idealized models were derived for the moment-rotation behavior of the WFJs: (i) a rigid-plastic model (RP) and (ii) a trilinear model (TL). The RP model, see Figure 7(a), presumes that no relative rotation exists for moments lower than the assumed maximum bending capacity, $M_{pl}/b$. Increasing rotation takes place at a constant $M_{pl}/b$ moment and up to a $\phi_{max}$ rotation at which failure occurs. The peak bending moment ($Ic-o$, $Ic-a$) or the upper boundary of the post-peak capacity ($If-o$, $If-a$, $V-d$, $V-s$) of the experimental $M$-$\phi$ curves was taken as $M_{pl}/b$; the maximum rotation of the series’ average plot was taken as $\phi_{max}$. In the TL model, see Figure 7(b), a three-segment ($OA$, $AB$, $BC$) piecewise linear function, composed of two ascending branches and one softening, descending branch, is used to represent the moment-rotation behavior. The average exp $M$-$\phi$ curves from Figure 6 and their corresponding $K_{pl}/b$, $M_{ult}/b$, $\phi_{ult}$ and $\phi_{max}$ are used as input parameters. The following assumptions were made: (i) the idealized initial rotational stiffness is equal to $K_{pl}/b$; (ii) the $M_{ult}/b$ and the corresponding $\phi_{ult}$ are respected – they define the softening initiation point $B$; (iii) the $\phi_{max}$ value is respected – it defines the $\phi$ coordinate of point $C$, taken as the softening end; (iv) $AB$ and $BC$ slopes are defined on the basis of the energetic balance between the idealized and the average $M$-$\phi$ experimental curves up to the $\phi_{ult}$ and $\phi_{max}$ rotations, respectively. The RP and the TL models for the $M$-$\phi$ relationships are represented in Figure 6(a)-(c).

In order to support the WFJ moment-rotation relationships calculated from the experimental results and validate the idealized models derived, two-dimensional finite element (FE) models were established of full-scale DS deck specimens subjected to three-point bending (Yanes-Armas et al. 2016a) and in-plane shear (Gürtler 2004) in their transverse-to-pultrusion direction. The experimental configurations modeled and the corresponding experimental results are shown in Figure 1. The FE-software SAP2000 and the Frame element were used. The web-to-flange connections were modeled as rotational springs whose behavior corresponded to the moment-rotation relationships shown in Figure 6, except for the external webs’ junctions in the three-point bending experiments (their behavior was not studied). Three models were established for each configuration to separately consider the experimentally-based $M$-$\phi$ curves (exp) and the idealized relationships (RP and TL). Displacement-controlled nonlinear static analyses considering geometric nonlinearities were performed.
Figure 1(c) shows the comparison of the measured load-displacement (P-δ) behavior of the deck under three-point bending and the corresponding numerical results obtained from the FE analysis (FEA). No differences were detected in the numerical results irrespective of the M-φ relationships used and a limited agreement with the experimental data was observed. Initial linear behavior up to a load of approximately 6 kN, matching the experimental results well, was predicted by the FE simulations. A slight nonlinearity followed, although it progressively deviated from the recorded load-displacement data, which showed a more pronounced stiffness reduction. At an approximately 14-kN load, sudden load drops of 20% can be observed in the numerical predictions, corresponding to two local failures appearing at the bottom junctions of the inclined 8th web and of the right external (9th) web. Local failure at the top junction of the 8th web, producing failure of the DS deck specimens at a 12.8-kN average load, was subsequently predicted, although at significantly higher loads (15.0–16.6 kN) than the experimental ones. The simulations showed a further increasing load-bearing capacity, differing from the experimentally observed sustained load capacity. These differences were attributed to shortcomings of the modeling, namely: (i) the initial thickness of the laminates remained unchanged throughout the simulations, whereas cracking propagating from the WFJs’ tensioned side towards the flange may reduce the effective thickness of the latter; (ii) the adhesively-bonded joints in the flanges were not considered; however, they may affect the deck’s global response; (iii) no experimental data were available for the M-φ relationships of the WFJs from the external webs. The in-plane shear configuration, where (ii) and (iii) do not apply (the flanges were mainly under compression and the rotational behavior of all the WFJ types involved had been studied) can thus serve better to assess the validity of the proposed moment-rotation relationships.

The measured and calculated load-displacement responses of the deck for the in-plane shear configuration are shown in Figure 1(d). The trends of the curves from the experiments and the numerical simulations compared well regardless of the M-φ relationships used, exhibiting initial linear behavior and subsequently pronounced nonlinear behavior up to the ultimate load, followed by a stopped descending branch showing decreasing load with large displacement development. Good agreement existed between the experimental results and the predictions from the FE models using the experimentally-based M-φ relationships (see FEA exp in Figure 1(d)): the calculated initial stiffness, taken as the slope of the P-δ curves, from the exp FE model overestimated the average experimental values by 13–17%; the predicted ultimate load differed from the experimentally obtained load by a maximum of 6%. This supported the validity of the experimentally-based M-φ relationships. Only a fair agreement was found when the idealized RP M-φ relationships were used in the modeling: the predicted initial stiffnesses overestimated the mean experimental ones by 37–46%; the experimental and the modeling ultimate load values differed by up to 10%; the simulated P-δ responses moderately represented the pre-peak nonlinear behavior. On the other hand, as good an agreement as with the exp M-φ curves was observed for the idealized TL M-φ relationships, as indicated by the almost coincident FEA exp and FEA TL P-δ plots. This suggested that the TL model could be effectively used to represent the moment-rotation behavior of the WFJs.

CONCLUSIONS

The rotational behavior of the three web-flange junction (WFJ) types from a pultruded GFRP deck with trapezoidal cell cross section (DS) was investigated. The following conclusions were drawn:

1. An experimental procedure to characterize the WFJ rotational behavior based on three-point bending and cantilever experiments conducted on the web elements was established.
2. The rotational stiffness, strength and failure mode of the WFJs differed depending on the web type, the WFJ location within the deck, the initial imperfections and the direction of the bending moment applied.
3. Numerical models of full-scale experiments conducted on DS deck specimens were developed, incorporating the experimental rotational responses of the WFJs. The validity of the calculated moment-rotation relationships was thereby demonstrated.
4. Two simplified relationships for the rotational responses of the WFJs were proposed: a rigid-plastic model (RP) and a trilinear model (TL). It was proven that the TL model can be successfully used to represent the actual moment-rotation behavior of the DS WFJs.

REFERENCES


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EXPERIMENTAL STUDY OF PULTRUDED GFRP BRIDGE DECK WITH DOUBLE CELLS

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ABSTRACT

A pultruded glass fiber reinforced polymer (GFRP) bridge deck with double cells has been developed and designed in laminate levels. Ultimate capacity tests to simulate vehicle loading were conducted to investigate its mechanical performance. The test results indicated that the load displacement relationship of such GFRP deck experienced from linear stage to nonlinear stage. In the first stage, specimens showed linearity until the damage of top slab. In the second stage, the load again but kept a nonlinear relationship with deflection. PM1 stayed in the second stage for quite a long period before failure with 36% load increasing. PM2 failed when the load increased to the peak value of the first stage. The failure mode of both specimens was longitudinal tearing crack on top slab caused by overlarge transverse tensile stress. PM1 had approximately 225% higher ultimate bearing capacity than PM2 due to different load positions.

KEYWORDS

GFRP, pultruded profile, bridge deck, model test.

INTRODUCTION

The fiber reinforced polymer (FRP) composites have gained more and more focus in the field of civil engineering, because of its high strength, light weight and non-corrodible nature (Bakis et al. 2002; He et al. 2012; Huang et al. 2015). Various applications of FRP bridge deck systems were proposed in terms of its advantage. Three major types of these applications are concrete bridge decks reinforced with FRP bars or FRP grids, all FRP composite bridge decks and hybrid FRP-concrete bridge decks (Xin et al. 2015a, 2015b).

Taking full advantage of FRP composites, all FRP composite bridge decks are lighter and easier to construct, and have huge potential use in the future bridge construction and rehabilitation. Park et al. (2005) developed an optimum design technique which can be applied to FRP bridge decks and performed wheel load tests on pultruded GFRP deck modules. The failure load was close to the computed value and three times larger than the axial design load specified in the specification. Lee et al. (2007) conducted wheel load tests on GFRP decks using two different lamination designs. In their study, it was found that the behaviour of the specimens was almost linearly elastic and showed brittle failure, and the design specification was sufficiently satisfied. Punching shear tests on GFRP tubular bridge deck specimens with multi cell cross-section or single cell cross-section was carried out by Prachasaree et al. (2009). The failure modes and punching shear capacities were obtained and compared with theory results. Researches above mainly focus on the macro properties of GFRP bridge decks. While the performance of GFRP profiles are affected by fiber volume fraction, fibre properties, resin properties, lamina thickness, lamina orientation, etc., it will be more significant to design FRP components from multiscale level. The integration of lamination design and structure design still needs further study.

A new pultruded GFRP bridge deck modular profile has been designed and manufactured from fiber/resin level to structure level in this paper. Double cells and double mortise and tenon joint structures were especially designed to accelerate the construction process and improve connection performance between deck modulus profile and deck modulus profile. The dimensions of unit module are shown in Figure 1(a). The laminate is designed based on classical laminate theory and the lay-up is shown in Figure 1(b). In order to investigate its mechanical
performance, ultimate capacity experiments were carried out, and responses including variation of displacements, stiffness, strains, cracks with the increasing of loading were investigated. All the results of experimental investigations in this study may provide reference for the design and construction of such type bridges.

EXPERIMENTAL PROGRAM

Test specimen

As shown in Table 1, the specimens were divided into two groups denoted as PM1 and PM2. The load position of PM1 was on top of the center web while PM2 was on top slab of a side cell at midspan. The load patch for PM1 was 250 mm × 250 mm based on vehicle model specified in the Standard of Loadings for the Municipal Bridge Design (CJJ 77-98), while the load patch for PM2 was reduced to 115 mm × 60 mm based on numerical calculation to consider tyre-to-deck contact pressure distribution (Sebastian et al. 2013). Before the test, the mechanical properties of top slabs, bottom slabs and webs of GFRP pultruded profiles were measured through a series of material tests. The test results were listed in Table 2.

Test setup and instrumentation layout

The specimens were simply supported at both ends using two hinges spaced at a distance of 1.8 m. The load was applied under displacement-control at a rate of 1 mm/min by a universal testing machine with loading capacity of 500 kN. Experimental setup along with LVDTs layout and strain gages layout are shown in Figure 2 and Figure 3 respectively. Load, deflection and strain signals were continuously recorded during testing using a high-speed data logger.

RESULTS AND DISCUSSION

Test results are summarized in Table 3. For each specimen, elastic capacity indicates the endpoint load value of linear segment of the load-displacement curve. Maximum deflection indicates the maximum deflection measured
at the time that the specimen reached ultimate bearing capacity. The elastic capacity, ultimate capacity and maximum deflection for PM1 (PM2) is 86.8 kN (35.6 kN), 117.7 kN (36.0 kN) and 15.27 mm (4.45 mm). Due to difference in load position, PM1 was more likely to bear the load by webs. In consequence, PM1 had approximately 225% higher ultimate bearing capacity than PM2.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Elastic capacity /kN</th>
<th>Ultimate capacity /kN</th>
<th>Maximum deflection /mm</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>PM1</td>
<td>86.8</td>
<td>117.7</td>
<td>15.27</td>
<td>Tearing crack at one side of top slab</td>
</tr>
<tr>
<td>PM2</td>
<td>35.6</td>
<td>36.0</td>
<td>4.45</td>
<td>Tearing crack at load point of top slab</td>
</tr>
</tbody>
</table>

Example: Failure process and mode

The failure mode of the specimens is shown in Figure 4. For PM1, a longitudinal crack appeared at the center of top slab at 86.75 kN. The specimen kept bearing and the crack propagated for the contribution of web after first crack, resulting in a new longitudinal crack at one side of top slab and several on outer webs at midspan. This new crack propagated and tore the top slab at 117.7 kN, leading to the failure of the specimen. For PM2, a longitudinal crack was observed at the load point of top slab at 18 kN, and more were observed on the outer web of loading side subsequently. Another top slab crack appeared beside the previous one at 35.6 kN leading to a sudden decrease of load. After then Cracks propagated while load increased nonlinearly. The specimen failed when one of the top slab cracks opened at 36 kN.

Load-deflection relationship

The load-deflection behaviour for the measuring points at midspan is shown in Figure 5. Both PM1 and PM2 exhibited elasticity in the beginning. The first damage of top slab reduced the load value by 50% and 25% respectively. Then the load-deflection curve behaved nonlinearly. In nonlinear stage, PM1 went through a long process before failure with load growth of 36% and noticeable increase in deflection. For PM2, there was little increase in load or deflection at the second stage.
Figure 5(a) indicates that the loading state of PM1 was almost symmetric in transverse direction. The center web bore more load and had larger deflection than outer webs. Figure 5(b) shows that there was noticeable torsion at midspan in PM2, and inverted camber in transverse direction in the bottom slab of loading side because of the rotation of web-flange joints. It is noted that cracks before 35.6 kN had almost no impact on the stiffness of PM2.

![Figure 4 Failure mode](image1)

**Figure 4 Failure mode**

**Figure 5 Load-deflection curve at midspan**

**Strain variations**

The deformation of top slab and bottom slab in the local bearing test was composed of two ways. The first was the longitudinal compressive or tensile deformation as top slab or bottom slab of the whole flexural member, and the second was the transverse and local deformation as elastic supported two-span continuous slabs which were fixedly connected to three webs. The overall longitudinal strain distribution for both specimens was similar to that of three-point bending beam, namely compressive strains on top slab and tensile strains on bottom slab. For PM1, the longitudinal strain distribution along section was almost symmetric, as shown in Figure 6(a). As for PM2, longitudinal strains gradually reduced to 0 from loading side to the opposite side because of eccentric load, and there was large local deformation at load point, as shown in Figure 7(a). The transverse strain distribution differed much more between two specimens. For PM1, the load was applied on top of center web, causing larger deflection at center web than outer webs. Consequently, as shown in Figure 6(b), transverse strains on top and bottom slabs were similar to the ones on top and bottom surfaces of an elastic beam, which is rigidly fixed at both ends and has vertical displacement at midspan. For PM2, the top slab deformed greatly and the deformation difference of each cell in transverse direction was more obvious, as shown in Figure 7(b).

Figure 8 shows the longitudinal strain distribution for outer webs at midspan. Both distributions were nonlinear. For PM1, longitudinal strains on two outer webs showed consistency. For PM2, the longitudinal strains on the web of loading side were much larger than those on the opposite web or web strains of PM1, indicating the web of loading side was in unfavourable state.
The strain results can be summarized as follows. For PM1, the maximum longitudinal strain at midspan was positioned at the center of top slab, and the maximum transverse strain at midspan was positioned on the outside of top slab. As for PM2, the maximum longitudinal strain at midspan was positioned on the outside of top slab, and the maximum transverse strain at midspan was positioned at the center of top slab. But for both specimens, the maximum strain measured was transverse strain on top slab, and the absolute values of top slab strains were generally higher than those of bottom slab strains. PM2 had a more serious state of local strain than PM1 under the same load level, and large strains of both directions were positioned on its top slab near the outer web of the loading side. So it is important to focus on the bearing condition of the outside of top slab as well as the outer web of the loading side under eccentric load during design.
CONCLUSIONS

A new pultruded GFRP bridge deck modular profile has been designed and manufactured from fiber/resin level to structure level in this paper. Local bearing test was carried out to evaluate the mechanical performance under wheel load. The following conclusions can be drawn from the present study:

(1) The failure mode of both specimens was longitudinal tearing crack on top slab caused by overlarge transverse tensile stress. PM1 had approximately 225% higher ultimate bearing capacity than PM2 due to different load positions.

(2) The loading process of local bearing test can be divided into two stages. In the first stage specimens showed linearity, until the damage of top slab. In the second stage the load rose again but kept a nonlinear relationship with deflection. PM1 stayed in this stage for quite a long time before failure, with an increase in load by 36%. PM2 failed almost as soon as the load recovered to the peak value of the first stage.

(3) Under the same load level, the average deflections of two specimens were close, but the strain states varied widely. For PM1, strain distribution was symmetrical in both longitudinal and transverse directions, and the maximum strain measured was positioned on the sides of top slab as transverse tensile strain. For PM2 which twisted under eccentric load, strains of the loading side were much larger than those of the opposite side whether on top, bottom slab or web. And the maximum strain measured was positioned at the center of top slab as transverse tensile strain. Therefore, during design, serious attention should be paid to transverse stress strain state of top slab when local loading, especially to the state on the loading side.

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FLEXURE STABILITY OF PULTRUDED GFRP I-SHAPES

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ABSTRACT

Pultruded glass fiber reinforced polymer (GFRP) members, are extensively used in industrial applications, particularly those subject to aggressive operating environments such as oil and gas exploitation. While many applications are ‘nonstructural’, increasingly, pultruded members are used in structural load bearing applications; typically as a ‘replacement’ for steel. The structural performance of pultruded GFRP members is strongly related to buckling behaviour. Although many designs treat GFRP buckling in a manner similar to hot- or cold-rolled steel, the behaviour is, in fact, notably different. Buckling and post-buckling behaviours of pultruded GFRP are affected by both the material’s anisotropy and the relatively low stiffness to strength ratio. The present research studied the flexure behaviour of pultruded I-shaped members having a variety of section geometries in order to identify the buckling behaviours: local flange buckling and lateral torsional buckling. The interaction between local and global buckling modes, which depends on the slenderness values related to each mode, was also identified. An extensive experimental program is reported and contrasted with proposed standards-based and computational (using the finite strip method) analyses. From these results the Finite Strip Method shows a good agreement with the experimental results, providing a reliable tool to predict the buckling behaviour of GFRP sections.

KEYWORDS

Pultruded GFRP, flexure tests, flange local buckling, lateral torsional buckling.

INTRODUCTION

The flexural performance of pultruded glass fibre-reinforced polymer (GFRP) structural members is often dependent on its buckling behaviour and structural resistance is affected by both the material strength and post buckling behaviour. Considered as an open thin-walled section, pultruded I-beams subject to flexure have three fundamental buckling modes: flange and/or web local buckling (FLB and WLB), lateral torsional buckling (LTB) and the interaction between FLB and lateral LTB. Buckling interaction depends on the slenderness associated with each buckling mode and its contribution to the reduction of the critical buckling load. Buckling interaction is rarely considered in extant studies of pultruded GFRP I-sections despite these elements often being used in applications in which such interaction is critical (Cardoso et al. 2015). Many researches including Kollár (2003), Sapkás et al. (2002) and Nguyen et al. (2014) have studied flexure stability (FLB and LTB) of pultruded GFRP beams using the classic plate theory approach, although such studies do not address interaction of FLB and LTB).

In addition, standards for pultruded GFRP are being developed. The 2010 American Society of Civil Engineers (ASCE) Pre-Standard for Load and Resistance Factor Design (LRFD) of Pultruded Fiber Reinforced Polymer (FRP) Structures (ASCE 2010) is presently being formalised into a forthcoming standard. The European Commission has proposed the Prospect for New Guidance in the Design of FRP - EUR 27666 (2016).

Moreover, computational analysis methods permit the buckling behaviour of the complete cross-section to be predicted avoiding approximate formulations based on plate theory. Besides the finite element method (FEM), the finite strip method (FSM) in which the cross-section is discretized into longitudinal strips (Shafer et al. 2006) has been shown to be very accurate for thin-walled longitudinal members. The FSM has fewer degrees of freedom than the FEM and is more easily implemented numerically since is not necessary to discretize the section in the longitudinal and transverse directions.

The objective of this paper is to study the flexure behaviour of pultruded GFRP I-shaped members having a variety of section geometries in order to identify the buckling behaviour: local flange buckling, lateral torsional buckling
and the interaction between the two. An experimental program is reported and contrasted with analytical and computational analyses promulgated ASCE (2010) and EUR 27666 (2016). The computational program CUFSM 4.05 (Li and Schafer 2010) was also used to obtain critical buckling loads and buckling modes.

**EXPERIMENTAL PROGRAM**

The experimental program was divided into two parts, with tests to identify the FLB and LTB behaviours, respectively. Both series of tests were conducted using the 900 kN capacity servo-hydraulic universal test frame shown in Figure 1.

Flange Local Buckling – Four-point Bending Tests

To evaluate FLB, five I-section geometries and four spans were defined. The geometries were selected according with the flange slenderness, b/2t, and the ratio of b/d. All profiles were fabricated from the same wide flange I-section: 6 x 6 x ⅛ (d x b x t in inch units; equivalent to 152.4 x 152.4 x 6.35 mm), having b/2t = 12. To obtain the other flange slenderness values, the flange tips (both compression and tension flanges) were simultaneously cut off using a band saw. The resulting profiles are summarised in Table 1. Simply-supported four-point flexure tests having spans of 2900 (two 1000 mm shear spans (i.e., a = 1000 mm) and a 900 mm constant moment region), 2600 (a = 900 mm), 2200 (a = 800 mm) and 1800 mm (a = 700 mm) were used.

In all tests, lateral support to prevent LTB was provided at both reactions points. For beams with low flange slenderness, additional lateral support was provided along the span, placed near the load points.

In order to obtain the onset and development of FLB, and the critical load, strain gages and digital images were used. The strain gages were placed on either side of the compression flanges in the constant moment region.

<table>
<thead>
<tr>
<th>specimen</th>
<th>section geometries d x b x t (mm)</th>
<th>section geometry</th>
<th>global slenderness, L₆/r</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>b/2t</td>
<td>b/d</td>
</tr>
<tr>
<td>FB6 or LTB6</td>
<td>152.4 x 152.4 x 6.35</td>
<td>12</td>
<td>1.00</td>
</tr>
<tr>
<td>FB5 or LTB5</td>
<td>152.4 x 127.0 x 6.35</td>
<td>10</td>
<td>0.83</td>
</tr>
<tr>
<td>FB4 or LTB4</td>
<td>152.4 x 101.6 x 6.35</td>
<td>8</td>
<td>0.67</td>
</tr>
<tr>
<td>FB3 or LTB3</td>
<td>152.4 x 76.2 x 6.35</td>
<td>6</td>
<td>0.50</td>
</tr>
<tr>
<td>FB2 or LTB2</td>
<td>152.4 x 50.8 x 6.35</td>
<td>4</td>
<td>0.33</td>
</tr>
</tbody>
</table>
Lateral Torsional Buckling – Three-point Bending Tests

To study LTB behaviour, the same cross sections used for FLB were used (Table 1). The simple span lengths and slenderness $L_{fb}/r$, given in Table 1, were selected to capture the LTB behaviour while mitigating to the extent possible FLB. Each specimen was tested over five unbraced clear span lengths as given in Table 1: 2896, 2438, 2134, 1829, and 1524 mm. Lateral support was only provided at the beam simple supports. Displacement transducers, strain gages, and digital images were used to capture the onset of LTB behaviour so as to determine the critical load.

Material

The pultruded profiles described above were made with fire retardant polyester resin and E-glass fibre from the same batch. Table 2 shows the average values (and COV in parentheses) of the mechanical properties obtained experimentally as well as those reported by the manufacturer and the minimum requirements from the ASCE Prestandard (2010). The transverse modulus, $E_T$ was determined with a non-standard test since the geometry of the pultruded profile did not permit coupons to be extracted having ASTM standard dimensions.

The material presents a high degree of anisotropy with the ratios of longitudinal to transverse modulus on the order to 2.6 and 3.3 for the flange and web, respectively. This kind of high anisotropy was also reported by Cunningham et al (2015).

<table>
<thead>
<tr>
<th>mechanical properties</th>
<th>test method</th>
<th>experimental values (COV)</th>
<th>manufacturer reported</th>
<th>ASCE (2010) minimum value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_L$ (MPa)</td>
<td>ASTM D3039</td>
<td>24490 (0.09) 26470 (0.04)</td>
<td>17200</td>
<td>20685</td>
</tr>
<tr>
<td>$E_L$ (MPa)</td>
<td>ASTM D6955</td>
<td>31219 (0.09) 31364 (0.05)</td>
<td>17200</td>
<td>20685</td>
</tr>
<tr>
<td>$E_T$ (MPa)</td>
<td>nonstandard</td>
<td>9444 (0.04) 8007 (0.07)</td>
<td>5500</td>
<td>5516</td>
</tr>
<tr>
<td>$G_L$ (MPa)</td>
<td>ASTM D3518</td>
<td>-             2882 (0.03) 3100</td>
<td>2758</td>
<td></td>
</tr>
<tr>
<td>$F_L$ (MPa)</td>
<td>ASTM D3039</td>
<td>300 (0.12) 310 (0.10)</td>
<td>207</td>
<td>207</td>
</tr>
<tr>
<td>$F_L$ (MPa)</td>
<td>ASTM D6955</td>
<td>325 (0.11) 227 (0.07)</td>
<td>207</td>
<td>207</td>
</tr>
<tr>
<td>$F_T$ (MPa)</td>
<td>nonstandard</td>
<td>85             -             48</td>
<td>48</td>
<td></td>
</tr>
</tbody>
</table>

RESULTS AND DISCUSSIONS

Flange Local Buckling – Four-point Bending Tests

The experimentally observed critical FLB moments and those obtained from various analytical and computational analyses are reported in Table 3. The experimental values represent an average of four tests from two different beams except those reported for FB3 which is based on three tests for the span of 2900 mm, and a single value for the span of 2200 due to a beam failure unrelated to buckling.

The experimental values obtained are in generally good agreement with the Finite Strip Method (FSM). The two ‘standard-based’ calculations – ASCE (2010) and EUR 27666 (2016) are conservative. In particular, the EUR 27666 (2016) yields predictions rarely exceed half of the experimentally determined critical loads. Both standard documents reportedly account for the interaction of FLB and LTB, which reduces the critical local flange moment. Table 3 reports the observed range of the ratio of analytical/computational predictions and experimental results.

All tests began with lateral support only at the reactions points. The interaction of FLB and LTB was expected for the beams with low flange slenderness, $b/2t \leq 6$; for these tests, additional lateral support was added along the span near the points of load application. In the initial test of FB5 ($b/2t= 10$), unanticipated interaction of FLB and LTB was observed for the 2900 mm span. A second test at a shorter span of 1800 mm was carried out to verify this result. These two tests are denoted with an asterisk in Table 3. Following these tests, additional lateral support was provided for all tests having $b/2t \leq 10$. The FB3 specimens exhibited significant LTB in which case the smallest FB2 tests, having $b/2t = 4$ and $b/d = 0.33$ were not conducted.

Figure 2 shows examples of the observed FLB for beams without the additional lateral support, FB6 and with the additional lateral supports, FB5.
Table 3 Average flange local buckling results.

<table>
<thead>
<tr>
<th>Span</th>
<th>Shear Span</th>
<th>Experimental</th>
<th>(M_{cr,FLB}) (N-m) from various calculations calculated with nominal section dimensions and (E_{L,t})</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>mm</td>
<td>mm</td>
<td>ASCE (2010)</td>
</tr>
<tr>
<td>FB6</td>
<td>2900</td>
<td>1000</td>
<td>9250 (0.06)</td>
</tr>
<tr>
<td>FB6</td>
<td>2600</td>
<td>900</td>
<td>9287 (0.03)</td>
</tr>
<tr>
<td>FB6</td>
<td>2200</td>
<td>800</td>
<td>10649 (0.06)</td>
</tr>
<tr>
<td>FB6</td>
<td>1800</td>
<td>700</td>
<td>11215 (0.08)</td>
</tr>
<tr>
<td>FB5</td>
<td>2900</td>
<td>1000</td>
<td>8175</td>
</tr>
<tr>
<td>FB5</td>
<td>2600</td>
<td>900</td>
<td>11396 (0.05)</td>
</tr>
<tr>
<td>FB5</td>
<td>2200</td>
<td>800</td>
<td>11325 (0.06)</td>
</tr>
<tr>
<td>FB5</td>
<td>1800</td>
<td>700</td>
<td>13531 (0.07)</td>
</tr>
<tr>
<td>FB5</td>
<td>1800</td>
<td>700</td>
<td>9538</td>
</tr>
<tr>
<td>FB4</td>
<td>2900</td>
<td>1000</td>
<td>14450 (0.06)</td>
</tr>
<tr>
<td>FB4</td>
<td>2600</td>
<td>900</td>
<td>14186 (0.06)</td>
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<td>FB4</td>
<td>2200</td>
<td>800</td>
<td>14040 (0.09)</td>
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<tr>
<td>FB4</td>
<td>1800</td>
<td>700</td>
<td>14655 (0.04)</td>
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<tr>
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<td>2900</td>
<td>1000</td>
<td>11750 (0.21)</td>
</tr>
<tr>
<td>FB3</td>
<td>2200</td>
<td>800</td>
<td>15000</td>
</tr>
</tbody>
</table>

* interaction of FLB and LTB observed.

Lateral Torsional Buckling – Three-point Bending Tests

Table 4 summarises the experimental and analytical/computational results of the LTB tests for all sections. The experimental values represent the average and COV of at least two tests. For the LTB tests, the position of the applied load needs to be taken into account since it may induce the LTB behaviour. Beams which are loaded above the shear center (top flange) are more susceptible to LTB and the critical load is lower than if the load is applied at or below the shear center (Bank 2006). The equation for critical LTB moment proposed by the ASCE Prestandard (2010), assumes that the load is applied at the shear center, while the equation propose by EUR 27666 (2016) specifically accounts for the position of the load. The critical load can be reduced nearly 30% when the beam is loaded at the top flange rather than the shear center, according with the tests results from Nguyen et al. (2014). All beams in this study are loaded at the top flange (Figure 1).

The ASCE (2010) standard calculations, yield generally unconservative predictions – more so for beams with greater flange slenderness, \(b/2t\). This behaviour is believed to be due to the interaction between the FLB and LTB observed in the experimental tests. A degree of the over-prediction from the ASCE Prestandard (2010) results from the fact that experimental loads were applied above the shear center.
The EUR 27666 (2016) predictions are unconservative for cases in which $b/2t > 8$, again, likely reflecting FLB/LTB interaction in the experimental beams. Figure 3 shows examples of the observed LTB from the tests of beams LTB6 and LTB3.

### Table 4 Average lateral torsional buckling results.

<table>
<thead>
<tr>
<th>Spec. M&lt;sub&gt;LTB&lt;/sub&gt; (N-m)</th>
<th>M&lt;sub&gt;cr&lt;/sub&gt; (N-m)</th>
<th>M&lt;sub&gt;cr&lt;/sub&gt; pred/exp</th>
<th>M&lt;sub&gt;LTB&lt;/sub&gt; pred/exp</th>
<th>M&lt;sub&gt;LTB&lt;/sub&gt; pred/exp</th>
</tr>
</thead>
<tbody>
<tr>
<td>LTB6 2896 3262 (0.10)</td>
<td>11775 3.32-3.72</td>
<td>7017 1.98-2.22</td>
<td>8388 2.6-3.22</td>
<td></td>
</tr>
<tr>
<td>LTB6 2438 4115 (0.05)</td>
<td>16243 3.52-4.06</td>
<td>9556 2.07-2.39</td>
<td>11378 2.47-2.84</td>
<td></td>
</tr>
<tr>
<td>LTB6 2134 4993 (0.06)</td>
<td>20940 3.96-4.45</td>
<td>12224 2.31-2.60</td>
<td>14404 2.72-3.06</td>
<td></td>
</tr>
<tr>
<td>LTB6 1829 6174 (0.02)</td>
<td>28173 4.44-4.62</td>
<td>16332 2.57-2.68</td>
<td>18809 2.96-3.08</td>
<td></td>
</tr>
<tr>
<td>LTB6 1524 7043</td>
<td>40164 5.65&amp;5.76</td>
<td>23141 3.25&amp;3.32</td>
<td>25354 3.56&amp;3.64</td>
<td></td>
</tr>
<tr>
<td>LTB5 2896 3212 (0.01)</td>
<td>7080 2.19-2.22</td>
<td>4311 1.33-1.35</td>
<td>5119 1.58-1.60</td>
<td></td>
</tr>
<tr>
<td>LTB5 2438</td>
<td>9673 2.38</td>
<td>5786 1.42</td>
<td>6920 1.70</td>
<td></td>
</tr>
<tr>
<td>LTB5 2134 3798 (0.06)</td>
<td>12397 3.26</td>
<td>7335 1.93</td>
<td>8775 2.31-2.31</td>
<td></td>
</tr>
<tr>
<td>LTB5 1829 5297 (0.03)</td>
<td>16588 3.02-3.25</td>
<td>9716 1.77-1.91</td>
<td>11357 2.10-2.27</td>
<td></td>
</tr>
<tr>
<td>LTB5 1524</td>
<td>5099</td>
<td>23532 4.62</td>
<td>13662 2.68</td>
<td>15968 3.13</td>
</tr>
<tr>
<td>LTB4 2438 3406</td>
<td>5225 1.48&amp;1.59</td>
<td>3220 0.91&amp;0.98</td>
<td>3796 1.07&amp;1.16</td>
<td></td>
</tr>
<tr>
<td>LTB4 2134 4123 (0.03)</td>
<td>6627 1.55-1.64</td>
<td>4019 0.94-0.99</td>
<td>4780 1.12-1.18</td>
<td></td>
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<tr>
<td>LTB4 1829 4173</td>
<td>8780 2.05&amp;2.16</td>
<td>5244 1.23&amp;1.29</td>
<td>6271 1.47&amp;1.54</td>
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<tr>
<td>LTB4 1524 4733</td>
<td>12344 2.61</td>
<td>7272 1.54</td>
<td>8684 1.83</td>
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<tr>
<td>LTB3 2896 1587 (0.36)</td>
<td>1886 0.98-2.57</td>
<td>1272 0.66-1.74</td>
<td>1403 0.73-1.91</td>
<td></td>
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<tr>
<td>LTB3 2438 1244 (0.27)</td>
<td>2467 1.41-2.33</td>
<td>1609 0.92-1.52</td>
<td>1824 1.04-1.73</td>
<td></td>
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<tr>
<td>LTB3 2134 1851 (0.03)</td>
<td>3070 1.62-1.73</td>
<td>1956 1.03-1.10</td>
<td>2257 1.19-1.27</td>
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<tr>
<td>LTB3 1829 1778 (0.04)</td>
<td>3990 2.16-2.33</td>
<td>2483 1.34-1.45</td>
<td>2913 1.58-1.70</td>
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<tr>
<td>LTB3 1524 2038 (0.001)</td>
<td>5506 2.70</td>
<td>3349 1.64</td>
<td>3980 1.95</td>
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<tr>
<td>LTB2 2896 402 (0.19)</td>
<td>767 1.58-2.33</td>
<td>580 1.20-1.76</td>
<td>581 1.20-1.77</td>
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<tr>
<td>LTB2 2438 575 (0.11)</td>
<td>962 1.44-1.79</td>
<td>700 1.05-1.30</td>
<td>725 1.08-1.35</td>
<td></td>
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<tr>
<td>LTB2 2134 955</td>
<td>1157 1.20&amp;1.23</td>
<td>817 0.85&amp;0.87</td>
<td>869 0.90&amp;0.92</td>
<td></td>
</tr>
<tr>
<td>LTB2 1829 886 (0.10)</td>
<td>1449 1.44-1.79</td>
<td>990 0.99-1.22</td>
<td>1082 1.08-1.34</td>
<td></td>
</tr>
<tr>
<td>LTB2 1524 1318 (0.07)</td>
<td>1918 1.41-1.62</td>
<td>1265 0.93-1.07</td>
<td>1423 1.04-1.20</td>
<td></td>
</tr>
</tbody>
</table>

(a) LTB 6 test  
(b) LTB3 test  
Figure 3 Lateral torsional buckling for the beams with the span of 2896 mm.

**CONCLUSIONS**

Existing standards (ASCE 2010 and EUR 27666 2016) are found to provide conservative predictions of the flange local buckling (FLB) critical moment. The Finite Strip Method (FSM) shows a good agreement with experimental results, providing a reliable tool to predict the local buckling behaviour of GFRP sections.

In the case of lateral torsional buckling (LTB), based on the experimental tests and analytical/computational studies, the ASCE (2010) is found to be unconservative in all flange slenderness. On the other hand, EUR 27666 (2016) provides unconservative predictions for beams with high flange slenderness which are expected to have FLB-critical behaviour, while for beams with flange slenderness less than 8, EUR 27666 (2016) yields relatively
accurate, if conservative predictions. The interaction between FLB and LTB observed in the experimental tests is the subject of an ongoing study.

ACKNOWLEDGMENTS

The authors wish to thanks Bedford Reinforced Plastics. The first author also thanks the financial support of CNPq (Conselho Nacional de Desenvolvimento Científico e Tecnológico), through scholarship 201799/2014-6. All tests were conducted in the Watkins-Haggart Structural Engineering Laboratory (WHSEL) at the University of Pittsburgh.

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ABSTRACT

GFRP pultruded profiles in GFRP–concrete hybrid structural elements present a remarkable structural capacity, not only for rehabilitation purposes but also in construction, for pedestrian bridges or lightweight bridge deck. In the literature, static, dynamic and creep experiments were performed on medium-scale structures. The connection between GFRP and concrete has been shown to be a shear connection since the friction is practically zero. Therefore, the shear connection test is used in this study to characterize both bolted and bonded connections. I-section GFRP pultruded profiles are connected to ordinary concrete by steel bolts in the first set and by epoxy in the second set. The experiments are monitored with a CCD-camera and digital image correlation (DIC) is used to evaluate the mechanical behaviour at the interface between GFRP and concrete. DIC allows a close examination of the deformation around the interface. Stiffness of the connection and distribution of the deformations around the area are compared and may be used to develop a mechanical model that will enable the simulation of GFRP–concrete elements on a larger scale.

KEYWORDS

Shear connection test, shear behaviour, GFRP-concrete connection, DIC.

INTRODUCTION

GFRP-concrete hybrid structural elements, consisting of GFRP pultruded profiles connected to concrete slabs, show a remarkable structural capacity not only for rehabilitation purposes, but also in the construction of lightweight bridge decks or footbridges. In fact, GFRP profiles have interesting mechanical and environmental properties such as high strength to self-weight ratio and good resistance to chemical agents. These properties make these elements suitable for structural applications for both building and bridges (Mendes 2011). Moreover, the association of these profiles with concrete compression elements makes a better use of both GFRP profile and concrete. Indeed, concrete elements increase the bending stiffness and the structural strength, reduce the deformations of GFRP profiles, and prevent the lateral buckling phenomenon.

GFRP-concrete hybrid structural elements have been studied in the literature. Static, dynamic (Correia 2007, 2009, El_Hacha 2012) and creep experiments (Gonilha 2014) were performed on medium-scale structures. Correia et al. 2009 investigated the connection between GFRP and concrete at the specimen scale. This connection has been shown to be a shear connection since the friction between GFRP and concrete is practically zero. Therefore, the push-out test (figure 1), similar to the standard push test in (EN 1994), is conducted in this study in order to characterize and compare bolted and bonded connections.

Data inferred from the force/displacement measurements on the test machine and by full-field digital image correlation (DIC) are used to accurately determine the stiffnesses of the both bolted and bonded connections. In future studies, results will be used to predict the mechanical behaviour of GFRP–concrete hybrid structural elements.
Figure 1 Dimensions of push-out test specimens (units are in mm)

SHEAR CONNECTION TEST

Test Setup

Push-out test specimens are fabricated according to the dimensions indicated in figure 1. In this study, two sets of 5 specimens are built. The first, POPB2, is connected by bonding while the second POPB3, is bolted. The composition of the concrete substrates was formulated in previous internal works. Compression tests done on 100 mm cubic specimens on the same day as push-out tests gave in case of bonding 48.4 MPa (± 3.7 MPa) and bolting 51.4 MPa (± 0.6 MPa).

For the bolted specimens, I-section GFRP profiles are prepared (cleaned, degreased), then the concrete blocks are cast in place successively. The two blocks of each push-out specimen are therefore not from the same mix (7 days apart). For bonded specimens, all concrete blocks are cast in one batch, and then the bonding is carried out after sandblasting and dusting down the surface to be bonded. The adhesive used is a bi-component epoxy Sikadur 31 CF. The thickness of the adhesive joint is not controlled, but estimated to be less than a millimetre. For bolting, 80 mm long M10 galvanized bolts with a class resistance of 6.8 are used. Four bolts are used for a specimen (two bolts per block).

Primary test results

Shear connection tests are performed on a Wolpert compression press under increasing monotonic loading until failure. The effort is applied on the top side of the I-section GFRP, the wood stiffener prevent local buckling. The crosshead speed is 0.6 mm/min. The effort and crosshead displacement are recorded. Force-displacement curves for all the specimens are given figure 2.

Excluding specimens with premature failure, bonded specimens show similar behaviours. After a loading phase while contacts are rearranged between the specimen and the testing machine, the force-displacement curve is almost linear up to failure. The shift between curves 1, 3 and 4 is due to the contact rearrangement at the beginning of the test. A load/unload cycle would probably overcome this issue. Among 5 specimens, two of them showed premature failure, probably due to mishandling or improper positioning of the specimen on the testing machine. For the 3 other ones, rupture occurs abruptly by shearing in the concrete near the interface and parallel to its plane (cohesive failure in the concrete). Before failure, no deformation is visible to the naked eye. Considering only the samples 1, 3 and 4, the average stress at failure is 7.95 MPa (± 0.9 MPa).
Bolted specimens also exhibit similar behaviour. At failure, the bolts are severely inclined, the slip between concrete and GFRP is visible. Failure comes after yielding then rupture of the bolts by bending. The average failure load is 26.3 kN (± 3.5 kN) per bolt.

Table 1 Properties of the optical sensor and the video lens

<table>
<thead>
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<th></th>
<th>mvBlueFOX 224G CCD</th>
<th>FUJINON 1 :1.4/16mm HF16SA-1</th>
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<tr>
<td>Model</td>
<td>CCD, gray scale</td>
<td>Focus</td>
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<tr>
<td>Resolution</td>
<td>1600 x 1200 Px</td>
<td>Iris</td>
</tr>
<tr>
<td>Max. frame rate</td>
<td>16 Hz</td>
<td>Manual</td>
</tr>
<tr>
<td>Sensor size</td>
<td>7.18 x 5.32 mm (1/1.8 '')</td>
<td>Focal length 16 mm</td>
</tr>
<tr>
<td>Pixel size</td>
<td>4.4 x 4.4 µm</td>
<td>Iris range 0.1 m – …</td>
</tr>
<tr>
<td>Exposure time</td>
<td>30 µs – 10s</td>
<td>Focusing range</td>
</tr>
</tbody>
</table>

Measurements with DIC

The DIC software Icasoft is used in this study to measure the displacement fields at the GFRP-concrete interface. Optical equipment specifications are given in Table 1. The specimens are prepared for the DIC: a black and white speckle pattern is applied to the zone of interest (ZOI). The optical sensor is placed at 45 ° with respect to the specimen at a distance of about 0.60 m as shown in figure 3. Thus, the visible faces of the profile and the concrete block are not in the same plane, this is taken into account when operating the results. By default, the following values are used for calculation (i) Pattern size, min = 25, max = 32 (ii) Grid step = 9 (iii) Precision = 1/100 pixel.
DIC RESULTS

Bonded connections

Out of the 5 specimens of POPB2 set, two tests are put aside due to premature rupture. A third test (bonded specimen 1) is also not exploitable with the DIC due to synchronization problems between the image acquisition and the crosshead displacement values. Then, only bonded specimens 3 and 4 will be discussed in the following. Vertical displacement fields for bonded specimen 3, for different loading values, are shown in figure 4. There is no clear discontinuity in the displacement field. The vertical displacements measured for the concrete block and the profile are not homogeneous. The displacement that is measured depends on the distance to the optical sensor. The further the point, the lower the measured displacement is. This observation is consistent; it indicates that the 2D field measurement is disturbed when the object is not included in a plane perpendicular to the optical axis.

![Figure 4 Bonded specimen 3, vertical displacements field for three loading values (left) 13.0 kN (center) 136.8 kN (right) 269.3 kN](image)

However, this aspect does not completely discredit the measurements. Here the measured displacements, rigid body displacements excluded, are comprised between -100 and 100 microns, while the size of a pixel is estimated to be 165 microns. The slip between concrete and GFRP, if there is any, is less than 200 microns at failure, and is not properly captured by the optical equipment available. The same observations are made for the bonded specimen 4. As a consequence, the behaviour at the interface between concrete and GFRP pultruded profile connected by bonding is regarded as infinitely stiff. Thus, we make the assumption of no slip at the interface until failure.

Bolted connections

The 5 specimens of POPB3 set are workable from the perspective of the DIC. For example, the ZOI of the bolted specimen 5 at the beginning and end of the test (last picture before failure) is given in figure 5. The deformation of the bolt is visible to the naked eye, it is strongly tilting downward. The slip measurement is done by comparing the vertical displacements between the points P1, P2 and P3 (situated on the profile) on one hand and B1, B2 and B3 (situated on the substrate) on the other hand (figure 5).

Vertical displacements fields for the bolted specimen 5, at different load levels, are shown in figure 6. We can observe the rigid body motion of the GFRP profile and the concrete block. The deformation of the specimen predominantly occurs at the junction, by deformation of the bolt.
At the beginning of the test, displacement is different from the upper section to the bottom of the concrete block. Figure 6.a, the difference is of the order of 0.3 mm, while the measured slip at the interface is 0.05 mm. This small difference is visible up to a load of 15.5 kN. Then from 15.5 to 18.6 kN, a vertical displacement jump appears between profile and concrete block. The measured slip is 0.1 mm. Subsequently, the slip is clearly observed between the two parts. Picture 3 (figure 6.c) is the last for which the image correlation can be made using a single ZOI. Over 48 kN, vertical slip strongly disturb the DIC measurement. It is then necessary to use two ZOI (Figure 6.d to 6.f).
A similar slipping behaviour is observed for all 5 specimens. The maximum slip is approximately 2.5 to 3.0 mm before failure, for a maximum load included between 23 and 30 kN per bolt. Thus, the behaviour at the interface between GFRP pultruded profile and concrete block bolted together will be modelled with a constitutive law that links slip to force. This law will be used to predict the mechanical behaviour of GFRP–concrete hybrid structural elements.

CONCLUSION

Two test series, 5 specimens per each, were tested by push out tests and the results were operated to compare both types of GFRP-concrete connection: bolting and bonding. The experimental approach using DIC to measure the relative displacement between the assembled elements shows promising results, with some limitations.

The study shows that the bonded connection demonstrates a very stiff behaviour with very small relative displacement recorded before failure. An average stress of 7.95 MPa (± 0.9 MPa) was measured at failure, without being able to accurately determine the corresponding slip. The DIC equipment used in this study indicates a relative displacement lower than ± 100 microns, but does not allow to define the slip more accurately. Further efforts are needed to investigate this point. In a first approach, the behaviour at the interface between concrete and GFRP pultruded profile connected by bonding is regarded as infinitely stiff. We can make the assumption that no slip occurs at the interface.

Bolted connections show a significant slip at the interface mainly due to bending of the bolts. The DIC technique allows measuring a homogeneous slip at the interface. The behaviour at the interface between GFRP pultruded profile and concrete bolted together will therefore be modelled by a constitutive law. This model will be used in interface modelling of GFRP-concrete hybrid structural elements.

REFERENCES


This paper presents an experimental investigation into the static performance of an all-composite modular composite beam under positive and negative bending moments. The deck was fabricated using lightweight pultruded GFRP box profiles adhesively bonded between two GFRP flat plates, adhesively bonded onto two FRP I-girders. A bidirectional pultrusion orientation was present within the deck structure, where the pultrusion (i.e. longitudinal fibre) direction of the box profiles were placed perpendicular to the pultrusion direction of the flat plates. This orientation improves structural performance by enhancing the junction between the box profiles and face plates to prevent premature cracking. In this paper, three different span lengths were investigated: a 4.5 m continuous span and 9 m and 6 m single spans. Composite action of the connections and the bidirectional deck was studied, along with shear lag under both positive and negative bending.

KEYWORDS
GFRP, modular construction, footbridge, composite system, adhesive bonding.
single span, and a 4.39 m continuous span. The spans were subjected to a uniformly distributed load (UDL). Bending stiffness, effective width and the degree of composite action provided by the adhesive connections (between the deck and girder) and by the transversely placed box profiles within the deck was investigated.

**EXPERIMENTAL INVESTIGATION**

**Material Properties**

Flat plates and FRP profiles were supplied by Exel Composites, Australia. The tensile properties were tested in accordance with ASTM D3039 (ASTM 2000). The material properties are given in Table 1 as an average of 10 specimens for the 6 mm-thick sections (flat plates, connecting plates and I-profiles) and 9 mm-thick sections. In addition, the in-plane shear modulus $G_{LT}$ for the 6 mm-thick sections was found using a 10° off-axis tensile test on coupons of size 250×25 mm. A two-part epoxy (R180 epoxy resin and H180 hardener supplied by Fiber Glass International) was used to adhesively bond the GFRP components together.

**Table 1** Material properties of 6 mm-thick and 9 mm-thick GFRP sections

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>6 mm-thick FRP</th>
<th>9 mm-thick FRP</th>
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<tr>
<td>Longitudinal elastic modulus $E_L$</td>
<td>GPa</td>
<td>22.99</td>
<td>24.63</td>
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<tr>
<td>Transverse elastic modulus $E_T$</td>
<td>GPa</td>
<td>10.32</td>
<td>10.03</td>
</tr>
<tr>
<td>Poisson’s ratio $\nu_{LT}$</td>
<td>-</td>
<td>0.30</td>
<td>0.31</td>
</tr>
<tr>
<td>Poisson’s ratio $\nu_{TL}$</td>
<td>-</td>
<td>0.15</td>
<td>0.14</td>
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<tr>
<td>Shear modulus $G_{LT}$</td>
<td>GPa</td>
<td>3.26</td>
<td>3.83</td>
</tr>
</tbody>
</table>

**Design of FRP composite system**

The 9 m modular FRP composite system is shown in Figure 1a. The sandwich deck was constructed using pultruded GFRP box-profiles adhesively bonded between two 6 mm-thick GFRP flat plates. Flat plates each had a width of 500 mm and a length of either 1.5 m or 3 m, placed in a staggered configuration as shown in Figure 1b. Adjacent flat plates were connected together using 6 mm-thick pultruded GFRP connecting plates, the pultrusion direction of which was placed parallel to that of the flat plates (along the $x$-axis, see Figure 1b). The box profiles had dimensions of 76×76×9.5 mm, with their pultrusion direction placed along the $y$-axis of the bridge (see Figure 1b), perpendicular to the pultrusion direction of the flat plates. The sandwich deck was adhesively bonded onto 203×203×6 mm pultruded I-beams spaced 900 mm centre-to-centre. The typical cross-section of the footbridge is shown in Figure 1c. The thickness of all bond lines was controlled by using 5 mm-long spacer wire of 0.7 mm thickness, placed at regular intervals along the adherend surfaces.

![Figure 1](image)

**Figure 1** a) Assembled composite system; b) plan view of composite system; c) typical cross-section of composite system (units in mm).

**Instrumentation and Experimental Setup**

Three span configurations were tested: 8.785 m simply supported span (Test 1, see Figure 2a), 6 m simply supported span (Test 2, see Figure 2b) and a 4.39 m continuous two-span (Test 3, see Figure 2c). All span configurations were tested under a 4.07 kPa uniformly distributed load, which corresponds to the SLS load based on design criteria for an FRP pedestrian footbridge (AASHTO 2008). Loading was applied by placing weights on the surface of the deck in 10 load steps. Loading was maintained for five minutes after each load step to collect strain and displacement data. Load levels were measured using load cells placed underneath each support. For Tests 1 and 2, two 25 kN load cells were placed at each support (LC1 to LC4, see Figures 2a-b). Test 3 had two additional 50 kN load cells at the central support (LC5 and LC6, see Figure 2c).
Figure 2 Plan view showing cross-sections and positions of displacement transducers for a) 8.785 m simply supported (Test 1), b) 6 m simply supported (Test 2) and c) 4.39 m continuous two spans (Test 3).

Four different cross-sections were instrumented for strain measurements. Strain gauges were placed along the depth of the deck and I-profiles in the longitudinal bridge direction at cross-section CS 1-1, which was 2.25 m from the end of the bridge. Strain gauges were also placed on the deck in the transverse bridge direction at this cross section. At cross sections CS 2-2 (situated 3 m from the end of the bridge) and CS 3-3 (situated 3.75 m from the end of the bridge), longitudinal strain gauges were placed along the upper flat plate. Finally, longitudinal strain gauges were placed along the depth of the structure and along the upper flat plate, and transverse strain gauges were also placed along the upper and lower flat plates of the bridge deck at cross-section CS 4-4 (situated 4.5 m from the end of the bridge).

Finally, deflections were measured using displacement transducers placed along the span of the bridge at either 1.5 m intervals (Test 1) or 1 m intervals (Test 2) on one side, with an additional transducer at the midspan position on the opposite side of the bridge as shown in Figure 2a and 2b. Deflections for Test 3 were measured along one span length as shown in Figure 2c, with maximum deflection measured by $D3$ situated 1.85 m from the support.

**RESULTS AND DISCUSSION**

**Load-deflection response**

The load-deflection responses for all test scenarios of the GFRP composite beam system are shown in Figure 3. Linear responses were observed for all specimens up to a load of 4.07 kPa. The bending stiffness $EI$ was calculated from the load-displacement curves using Euler beam theory, found to be 6.61×10$^{12}$ Nmm$^2$ for Test 1 and 6.02×10$^{12}$ Nmm$^2$ for Test 2 (with a difference less than 10%). However, the bending stiffness calculated for Test 3 was 2.58×10$^{12}$ Nmm$^2$, which was about 60% lower than those calculated for Tests 1 and 2. This might be due to the effect of shear deformations in the 4.39 m span of Test 3. The effects of shear deformations can be taken into account using Timoshenko beam theory. One span of a continuous beam can be considered as a propped cantilever. Deflection $z$ is calculated using Eq. 1 (Wang et al. 2000):

$$z = z_E + \frac{1}{GAK_s} \left[ \frac{3Lwx}{8} - \frac{wx^3}{2} \right] + \frac{3}{GAK_s(1 + 3\omega)} \frac{x}{L} \left( \omega + \frac{1}{2} - \frac{x^2}{6L^2} \right) \left( \frac{wL^2}{8} \right)$$  

(1)
where $G$ is the shear modulus (taken as 3.26 GPa from Table 1), $A$ is the area of the section (taken as the area of the I-profiles as it is assumed that the shear stresses are carried by the I-profiles only), $L$ is the span length, $w$ is the uniformly distributed load and $x$ is the position along the span measured from the support. A shear correction factor $k_s$ was calculated based on an I-section as 0.28 from Cowper (1966). $z_E$ is the deflection based on Euler beam theory given as Eq. 2 and $\Omega$ is a factor calculated using Eq. 3. For Test 3, the bending stiffness $EI$ calculated from Eq. 1 was $5.5 \times 10^{12}$ Nmm$^2$, which was 9% less than that found for Test 2. Shear deformations contributed to 51% of the overall deflection.

$$z_E = \frac{wx}{48EI} \left( \frac{L^3}{3} - 3Lx^2 + 2x^3 \right)$$ (2)

$$\Omega = \frac{EI}{GAK_s}$$ (3)

Figure 3 Midspan load-deflection curves for of 8.785 m (Test 1), 6 m (Test 2) and 4.39 m (Test 3) spanning composite systems under a uniformly distributed load of 4.07 kPa.

**Longitudinal strain distribution along specimen depth**

The longitudinal strain distribution along the depth of the composite system is shown in Figure 4 at 4.07 kPa. The strain distributions were obtained at midspan (CS 4-4, see Figures 2a and 2b) for Tests 1 and 2, and within the positive bending region (CS 1-1 in Figure 2c) for Test 3. As bending moments were low in the negative bending region for Test 3, the resulting strain values were small (less than 100 μstrain) and their accuracy was not sufficient. Therefore strain distributions were not presented for the negative bending region (CS 4-4) for Test 3. As shown in Figure 4, the axial strain distributions were linear from the lower flange of the I-profiles to the lower flat plate, indicating that full composite action was provided by the adhesive bond between the I-profiles and the deck. In addition, the strain distributions were also linear from the lower flat plate to the upper flat plate, indicating that the transverse webs of the deck provided full composite action for the positive bending regions.

Figure 4 Longitudinal strain distribution along the depth at 4.07 kPa.

Full composite action provided by the transversely placed box profiles within the deck was not observed in a previous study by Satasivam and Bai (2015) that also utilised built-up decks with bidirectional orientations. In that study, partial composite action was observed between the upper and lower flat plates as a result of the weak in-plane shear stiffness of the transversely oriented webs. The thickness of the box-profile webs in Satasivam and Bai (2015) was 6 mm, with box-profiles spaced 105 mm centre-to-centre. In comparison, the web-thickness of the
box-profiles in this study was 9.5 mm and spaced at 252 mm centre-to-centre. These strain distributions indicate that a thicker transverse web can provide sufficient shear stiffness. Hence, built-up pultruded sandwich decks with bidirectional pultrusion orientations can show full composite action between the upper and lower decks, depending on the in-plane shear stiffness offered by the web sections.

**Longitudinal strain distribution along specimen width**

Figure 5 shows the longitudinal strain distribution of the upper flat plate along the specimen width for Test 1 (8.785 m), measured from the centre of the beam width. At CS 2-2, the absolute values of compressive strains along the upper plate were lowest at positions where connecting plates were situated compared to the compressive strains at the centre of the beam (y=0 in Figure 5a). At CS 3-3, which does not have connecting plates, the compressive strains were relatively uniform along the width of the beam. At CS 4-4, the absolute values of compressive strains were lowest at the centre of the beam width where connecting plates were situated. These changes in strain distribution were therefore due to the presence of connecting plates. The connecting plates increased the stiffness of the flat plate locally (because of the increased thickness), resulting in a lowered compressive strain. The change in the strain distribution along the deck width caused by the change in thickness of the flat plates (due to additionally bonded connecting plates) may therefore affect the determination of the effective width. Such behaviour was also observed for Test 2 (6 m).

![Figure 5 Longitudinal strain distributions along the width of the upper flat plate for Test 1 at 40%, 60%, 80% and 100% of 4.07 kPa loading measured at a) CS 2-2, b) CS 3-3 and c) CS 4-4.](image)

The transfer of loads between adjacent flat plates joined together with connecting plates was measured using strain gauges L9, L10 and L18 at CS 2-2 (as shown in Figure 6a), and L31, L32 and L36 at CS 4-4 (see Figure 6b). The stresses at these positions were obtained from measured strain results and are summarised in Table 2. It can be seen that loads were directly transferred across the upper flat plates, considering the similar stress values on either side of the connecting plate (i.e. from L9 to L10 for CS 2-2 and from L31 to L32 for CS 4-4). This also indicates that the changes in effective width shown in Figure 5 are mostly due to the increase in plate thickness rather than insufficient load transfer at the connection plates.

![Figure 6 Strain gauge positions for connecting plates at a) CS 2-2 and b) CS 4-4.](image)
Table 2 Longitudinal stresses on upper flat plates and connecting plates

<table>
<thead>
<tr>
<th>Cross-section</th>
<th>Strain gauge position</th>
<th>Longitudinal stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Test 1</td>
</tr>
<tr>
<td>CS 2-2</td>
<td>L9</td>
<td>-9.9</td>
</tr>
<tr>
<td></td>
<td>L10</td>
<td>-8.1</td>
</tr>
<tr>
<td></td>
<td>L18</td>
<td>-5.0</td>
</tr>
<tr>
<td></td>
<td>L31</td>
<td>-13.3</td>
</tr>
<tr>
<td>CS 4-4</td>
<td>L32</td>
<td>-15.2</td>
</tr>
<tr>
<td></td>
<td>L36</td>
<td>-6.2</td>
</tr>
</tbody>
</table>

CONCLUSIONS

A modular GFRP composite system was developed in this paper with a built-up sandwich deck with bidirectional pultrusion orientations, and then adhesively bonded to and supported by pultruded GFRP I girders. This composite system was investigated under static loading and different span configurations with positive and negative bending scenarios. The following conclusions can be drawn:

1) The load-deflection curves showed linear responses up to a load of 4.07 kPa. Shear deformations are negligible in the 8.785 m (Test 1) and 6 m (Test 2) spanning composite systems. However, shear deformations account for 51% of total deflections in the 4.39 m (Test 3) spanning continuous system.

2) Full composite action can be achieved at the adhesive connection between the I-beams and the lower flat plate. Full composite action can also be achieved within the FRP deck between the upper and lower flat plates because the transverse webs provide sufficient shear stiffness.

3) The presence of connecting plates results in a localized increase in the thickness of the upper flat plate, resulting in a reduction in compressive strains in the longitudinal direction. This may result in a change in the determination of the effective width.

4) Full load transfer occurs between adjacent flat plates joined together with connecting plates, indicating that changes in effective width do not arise as a result of insufficient load transfer at connecting plates.

Taking advantage of the light weight of such a modular GFRP composite beam, this structure is used as a footbridge to investigate human-induced vibration at Monash University. It is also expected that such a modular approach of structural assembly may be considered for potential footbridge construction.

ACKNOWLEDGMENTS

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REFERENCES

NONLINEAR ANALYSIS OF FRP CABLE NET STRUCTURES

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3 School of the Civil Engineering and architecture, Henan University, China.

ABSTRACT

Cables are widely used in large-span structures. A FRP cable net structure was constructed at first, then the anchorage system, joints and construction steps were also detailed. An improved rigid cable method was provided to calculate the internal forces of FRP cable net structures. After nonlinear analysis based on the improved rigid cable method, the force performances of the FRP cable net structure were discussed. The FRP cable net structures are applicable with enough structural strength and stiffness. A very light weight is worthy of being noted. A shortcoming of the FRP cable net structures is that its displacement is large relatively. The reason is the Elastic modulus of FRP is small relatively. This shortcoming can be remedied by improving structural design. The FRP cable net structures are specially suitable in long-span structures due to high strength-to-weight ratio and easy construction.

KEYWORDS

FRP, cable net structures, nonlinear analysis, improved rigid cable method.

INTRODUCTION

Cables are widely used in large-span structures(Richard,2002; Gould, 1998). Except of the graceful geometrical configuration, cables have excellent load carrying performances. Cables can make full use of the bearing capacity of tensile materials for there is only the tensile axial force in a cable. In tropical regions of India, Africa and South America, vines cables were used to build suspension bridges long time ago. Vines are weak material but those suspension bridges were strong and stable enough to support Spanish armies and their horses.

Steel, the main material nowadays, contributes to the application of cables greatly(Billington,1990; Farshad,1992; Tedesco,1991; Dong,2012). Steel cables have high strength-to-weight ratio, so they are fitful to go across large distance. Transmission tower lines all over the world are good examples. Steel cables were used to build bridges in ancient times. It has been over 400 years since ancient Chinese build the Luding bridge in Sichuan province in 1696. Steel cables were only used in some important or special structures in ancient time for the quantity of steel was rare. Till twenty century steel cables were widely used in all kinds of large-span structures, such as suspension bridges, roofs of stadium, arena, airport terminal and exhibition halls(Robert,1999; Karbhari,2000; Teng,2002; Feng,2007).

Cable net structures, only made of cables, are the best examples to embody the characters of grace and lightness(Reis,2008; Sharaf,2010; Stone,2001; Zhu,2009). World famous constructions, including the Raleigh Arena(USA 1953), the Exhibition Hall at Portmunn(German,1956) and Pengrowth Saddle Dome(Canada, 1983) are all landmarks. However the weight of steel cables are still large when they stride over larger distance. And corrosion and fire-resistant are also steel cables’ shortcomings.

The Fibre Reinforced Polymer(FRP) composites bring new opportunity to the use of cables. FRP composites are being promoted as 21st century materials because of their high strength-to-weight ratio, superior corrosion resistance and excellent thermo-mechanical properties. FRP composites are also greener material than conventional material such as steel. The use of FRP composites in civil engineering can improve innovation, reduce life-cycle costs and provide longer service lives(Dawood,2010; Vaidya,2010; Mousa,2012).
In this paper, a FRP cable net structure was constructed and the anchorage system, joints and construction steps were also detailed. After nonlinear analysis based on the improved rigid cable method using ANSYS, the force performances of FRP cable net structure were discussed comparing with steel cable net structure.

**CONFIGURATION OF FRP CABLE NET STRUCTURES**

**FRP laminates**

FRP laminates are used as cables instead of steel wire ropes in cable net structures. Aslan 400 CFRP laminates, made by Hughes Brothers Inc USA, are recommended. The width of a laminate is 100mm and the thickness is 1.4mm. Tensile and modulus properties are measured per ASTM D3039, the standard test method for tensile properties of polymer matrix composite materials. Aslan 400 mechanical properties are provided in Table 1.

<table>
<thead>
<tr>
<th>Properties</th>
<th>CFRP laminates</th>
<th>steel wire ropes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Guaranteed tensile strength</td>
<td>2400 MPa</td>
<td>1760 MPa</td>
</tr>
<tr>
<td>Tensile modulus of elasticity</td>
<td>131 GPa</td>
<td>210 GPa</td>
</tr>
<tr>
<td>Ultimate stain</td>
<td>1.87</td>
<td>1.4</td>
</tr>
<tr>
<td>Density (g/cm$^3$)</td>
<td>2.126</td>
<td>7.85</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.25</td>
<td>0.25</td>
</tr>
</tbody>
</table>

In comparison, stainless steel cable mechanical properties are in the list too. The stainless steel wire rope type 304, provided by E-rigging.com is the most common type of cables. From the information in table 1, the weight CFRP laminates is about one fifth of steel wire rope when they resist the same load.

**Structural model**

A FRP cable net structure was built using the software ANSYS, as shown in Figure 1. The double curved roof was designed for it was good to drain off water and reduce the horizontal reactions at supports. It is also graceful with curved configuration. The plane projection is a rectangular whose size is 150m(length) $\times$ 90m(width). The cables along the width are the main load bearing members under downward loads, so it bends downward. And the cables along the length bend upwards and become the main load bearing members under upward loads of wind (Figure 2).

![Figure 1 A model of FRP cable net structure](image1)

![Figure 2 Main bearing cables under downward and upward loads](image2)

The sag at the middle point of each cable is 15m, which is determined after many trial calculations by considering architectural factors and inner forces. The grid size of cable net structure on planar projection is 3m$^2$. This size is fitful to pave roof surface members. The initial shape of cables were set to be art, while final shape under pre-stress and loads can be obtained by form finding analysis. A ring beam made of concrete was set on concrete columns to provide supports for cables.

**Anchorage system**

FRP’s high ratio of axial tensile strength to lateral compressive strength, approximately 20 to 1, is the main challenge to enlist an anchorage system that will ensure full development of FRP strength. The ends of each FRP laminate need to be anchored on the ring beam by hinge supports. Some anchorage systems for FRP sheets have been proposed, and in these systems FRP sheets are adhesive on concrete surface and they do not lead to the hinge joints at the ends of laminates. A tentative hinge joint scheme, where the FRP laminate is clamped between two stiff plates, was provided, however, it is difficult to apply pre-stress to cables.
A new hinge joint scheme between a strip and a ring beam is shown in Figure.3. Two steel plates are fixed on the ring beam. A bolt is set between two steel plates. The FRP laminate binds around the bolt and bonds using adhesives Pilgrim EM5-2 GEL. The pre-stress can be applied by rotating the bolt to shorten the laminate. Then a piece of film made of carbon fibre bundles the joints area to strengthen the connection. The film wraps the bolt and laminate in-side.

Figure 3 The hinge joint anchorage system

NONLINEAR ANALYSIS

FEM model and load cases

A structural model is provided in Figure 1. The FEM software ANSYS is adopted to perform the analysis of the FRP cable net structure and the steel cable structure. Geometrical nonlinear and large displacement analysis are required. The cables are simulated using three-dimensional (3D) Link10 element. The ring beam and columns are simulated using 3D Beam4 element. Two load cases are considered. In load case 1, the pre-stress, self-weight and downward 1kN/m² uniform load are included. In load case 2, the pre-stress, self-weight and upward 1kN/m² uniform load are included.

Optimization of pre-stress

The cable net structure requires a pre-stress to complete configuration. At present, there are three cable pre-stressed optimization methods for cable net structures. They are geometric method, the tangential balance method and the rigid cable method. Now the rigid cable method is the most convenient for it has clear concept and easy to operate by applying initial strain. However, the pre-stress losses caused by the structural deformation is not included. An improved rigid cable method is provided here. At first, according to the rigid cable method, the elastic modulus of each cable is amplified 100 times. And static calculations are carried out using ANSYS to obtain the axial force of each cable element, which is recorded as \( N_i \). The initial strain is:

\[
\varepsilon_i = \frac{N_i}{E_i A_i} \quad (1)
\]

where, \( \varepsilon_i \) is the initial strain of cable element \( i \), \( E_i \) is the elastic modulus of cable element \( i \), and \( A_i \) is the section area of cable element \( i \).

Then the elastic modulus of each cable is changed to go back original values and initial strains are applied to cable elements. Compute initial strain based on \( N_i \), then static calculations are carried out again using ANSYS to obtain the axial force of each cable element, which is recorded as \( \bar{N}_i \). The pre-stress loss of each cable element can be obtain, which is recorded as \( \Delta N_i \):

\[
\Delta N_i = N_i - \bar{N}_i \quad (2)
\]

Then the temperature stress is adopted to offset the pre-stress loss. Since

\[
\Delta N_i = \frac{\Delta L_i}{E_i A_i L_i} \quad (3)
\]

\[
\Delta L_i = T_i \alpha L_i \quad (4)
\]

Where \( L_i \) is the length of cable element \( i \), \( \Delta L_i \) is the change of length of cable element \( i \), \( T_i \) is the temperature of cable element \( i \), and \( \alpha \) is the material line expansion coefficient of temperature.

So, we obtain

\[
T_i = \frac{\Delta N_i}{E_i A_i \alpha} \quad (5)
\]

Therefore, if the temperature \( T_i \) is applied to the corresponding cable element, pre-stress loss \( \Delta N_i \) can be offset by the temperature stress to offset. The concept of above improved rigid cable method is to compensate pre-stress loss using temperature stress.
The optimized pre-stress force of each FRP cable along the direction of length and width were 591kN and 324kN respectively. While the value for each steel cable along the direction of length and width were the same when we analyzed the steel cable net structure.

**Results of displacement and stress analysis**

Based on the previously described finite-element model, numerical calculations with ANSYS were conducted. The results of nonlinear static analysis are summarized in Figure 4 and Figure 5.

![Figure 4 Stress of the FRP cable net structure](image1)

![Figure 5 Displacement of the FRP cable net structure](image2)

It can be observed that the maximum stress of FRP cable net is 2248 MPa, which shows the axial stretch force is the main internal force. The maximum vertical displacement of the FRP cable net is 1.625 m (Figure 5). In comparison, the maximum vertical displacement of the steel cable net is only 0.884 m. This indicates that the structural stiffness of FRP cable net is weak because the Elastic modulus of FRP is small relatively.

Geometrical nonlinear and imperfections often lead to conservative results of Eigen-value buckling analysis. And nonlinear buckling analysis can satisfy the real-world situation such as large deflection and initial imperfection better. In respect to the geometric imperfection, the first-order buckling mode shape was used for the configuration of the most dangerous geometrical imperfection structure which is similar to the first-order buckling mode shape. The maximum deformation value of geometric imperfection was 1/1000 of the span. The value is according to the Chinese Code for Acceptance of Construction Quantity of Steel Structures [GB50205-2001], and it is consistent with the Code for Standard Practice for Steel Buildings and Bridges [AISC 2005]. The imperfection was considered in ANSYS by updating geometry after Eigen-value buckling analysis.

Analytically predicted results for the FRP cable net structures are presented. The results are obtained from finite element models of geometrically perfect structures and structures that include initial geometric imperfections. It can be observed that the influence of geometrical nonlinear is dramatic. The analysis considering large deformation is necessary. It is also can be observed that the buckled mode shape of geometrical nonlinear analysis does not agree with the first-order mode shape from the Eigen value analysis. Since the Eigen value analysis predicts the theoretical buckling strength of a structure which is idealized as elastic and linear. However, the applied loading incrementally causes a large change in displacement during the modification and interaction of structural stiffness matrix. Therefore, the first-order mode shape from the Eigen value analysis may be different from the nonlinear buckled mode shape. It is necessary to run nonlinear buckling analysis to obtain the actual displacements, forces, and reactions.

**CONCLUSIONS**

Cable net structures made of FRP were presented in this paper. Based on the ANSYS program, the geometrical nonlinear analysis is carried out. The connection is also investigated. The main conclusions from this study can be summarized as follows:

1. Proposed FRP cable net structures have the advantages in terms of light weight, mass production, ease of assembly, lower skill required for field construction, good quality control and worker safety, and corrosion resistance.
2. The FRP cable net structures are applicable with enough structural strength and stiffness. A very light weight is worthy of being noted.
A shortcoming of the FRP cable net structures is that its displacement is large relatively. The reason is the Elastic modulus of FRP is small relatively. This shortcoming can be remedied by improving structural design. A structural system combined of FRP and steel members is worth to study further.

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EXPERIMENTAL INVESTIGATION ON THE FLEXURAL BEHAVIOUR OF PULTRUDED GFRP BEAMS FILLED WITH DIFFERENT CONCRETE STRENGTHS

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Email: majid.alzaidy@gmail.com, majid.muttashar@usq.edu.au

ABSTRACT

Glass fibre reinforced polymer (GFRP) pultruded profiles are being increasingly used in the construction industry due to their numerous advantageous over the conventional materials. However, most pultruded GFRP sections fail prematurely without utilising their high tensile strength due to their thin-walled sections. As a result, several hybrid systems made out of GFRP profiles and concrete as a filler material have been proposed in order to enhance their structural performance. Most of these studies utilised high strength concrete wherein the additional cost does not justify the enhancement in the stiffness and strength of the infilled GFRP profiles. This paper presents an experimental investigation on the effect of the compressive strength of concrete infill on the flexural behaviour of beams with a view to determine a lower cost infill for GFRP profiles. Pultruded GFRP square beams (125 mm x125 mm x 6.5mm) were filled with concrete having 10, 37 and 43.5 MPa compressive strength and tested under static four-point bending. The results showed that the capacity of the filled beam sections increased by 100 to 141% than the hollow sections. However, the compressive strength of the concrete infill has no significant effect on the flexural behaviour of the beams. The increase in concrete compressive strength from 10 to 43.5 MPa increased the ultimate moment by only 19% but exhibited an almost same flexural stiffness indicating that a low strength concrete is a practical solution to fill the GFRP profiles.

KEYWORDS

FRP, hybrid beams, concrete infill, low-strength concrete.

INTRODUCTION

Fibre reinforced polymer (FRP) composite materials have recently been used in the construction industry especially in corrosive environment. One of the innovative applications is the concrete filled FRP tubes (CFFTs) due to the relatively low elastic modulus as well as thin-walled sections of the FRP tubes. Extensive research was carried out on CFFTs as columns, but relatively limited research was conducted on CFFTs as beams (Fam & Rizkalla 2001; Mirmiran & Shahawy 1996; Mohamed & Masmoudi 2010; Ozbakkaloglu 2012). Most of these research concentrated on the ability of this system to take advantage of the confinement provided by the composite tube to the concrete core. The growing incorporation of CFFTs in compression application let to expand the investigation of the feasibility of this system for beam applications. In the last decade, several research efforts were made to investigate the flexural behaviour of concrete filled FRP tubes. Roeder, Lehman and Bishop (2010) concluded that the local buckling resistance of the FRP tubes increases due to the contribution of the concrete infill. Similarly, Fam and Rizkalla (2002) carried out experimental investigation on the flexural behaviour of hollow and filled GFRP circular tubes with a range of concrete infill strength between 30 and 60 MPa. They concluded that the flexural behaviour is affected to some extent by the concrete compressive strength. In addition, Mirmiran and Shahawy (1996) carried out experimental investigation to compare the flexural behaviour of concrete filled FRP tubes and the conventional reinforced concrete beams. The results showed the performance of the concrete filled beams is comparable or better than the conventional reinforced concrete beams. Chen and El-Hacha (2010) studied the flexural behaviour of FRP sections filled with ultra-high performance concrete (138 MPa). They concluded that the concrete core provides lateral support for the section and it prevents compressive flange buckling at higher loads.
Most of the previous research utilised high strength concrete as a suitable filling material of FRP where the additional cost does not justify the enhancement in the stiffness and strength of the infilled GFRP profiles. This paper presents an experimental investigation concerning the flexural behaviour of GFRP tubes filled with concrete having different compressive strength. The main objective of this study is determining a lower cost infill for GFRP profiles.

**EXPERIMENTAL PROGRAM**

**Material Properties**

Glass fibre reinforced polymer (GFRP) pultruded tubes and concrete are the main materials used in this study. Details of these materials are provided in the following sections.

**GFRP tubes**

A square Pultruded GFRP sections (125 mm x 125 mm x 6.5 mm thickness) produced by Wagner’s Composite Fibre Technologies (WCFT), Australia were used in this study. The tubes were produced using pultrusion process using vinyl ester resin with E-glass fibre reinforcement. Burnout test was conducted according to ISO 1172 (ISO 1172 1996) where the density and the fibre volume fraction are found to be as 2050 kg/m³ and 78% by weight, respectively. The stacking sequence of the plies is [0°/+45°/0°/-45°/0°/+45°/0°/-45°/0°/+45°/0°], where the 0° direction accords with the longitudinal axis of the tube. The mechanical properties (elastic modulus and shear modulus) of GFRP sections were determined previously by Muttashar et al. (2015). In addition, coupon tests were conducted to find the compressive and tensile strength of the sections. Table 1 shows the mechanical properties of the tested samples.

<table>
<thead>
<tr>
<th>Material property</th>
<th>Symbol</th>
<th>Property value</th>
<th>unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>ρ</td>
<td>2050</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Tensile stress</td>
<td>σₜ</td>
<td>596</td>
<td>MPa</td>
</tr>
<tr>
<td>Tensile strain</td>
<td>Ɛₜ</td>
<td>16030</td>
<td>Microstrain</td>
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<td>Compressive stress</td>
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<td>MPa</td>
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<tr>
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<td>47.2</td>
<td>GPa</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>G</td>
<td>4</td>
<td>GPa</td>
</tr>
</tbody>
</table>

**Concrete**

Three different concrete strengths were used as concrete infill of the tubes. Five plain concrete cylinders have been sampled from each batch and cured under the same conditions as the beam sections. The 28 days average compressive strength was of 10, 37.5 and 43.5 MPa, respectively.

**Test specimens**

Three GFRP hollow sections and six GFRP filled sections were used in this study to investigate the flexural behaviour. The total length of the tested beams was 2000 mm. The details of the tested specimens are shown in Table 2. Considering the strength of the filled concrete, the specimens were identified using the code listed in the Table. The term BH indicates the hollow geometry, while, BC-10, BC-37 and BC-43 represent the beams filled with concrete having 10, 37 and 43 MPa compressive strength, respectively.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Description</th>
<th>Concrete strength MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>BC-10</td>
<td>10</td>
<td></td>
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<tr>
<td>BC-37</td>
<td>37</td>
<td></td>
</tr>
<tr>
<td>BC-43</td>
<td>43</td>
<td></td>
</tr>
</tbody>
</table>
Test setup and instrumentation

Four-point bending test was performed over a simply supported clear span of 1800 mm following the procedure in ASTM D7250 (ASTM D7250 2006). The specimens were tested on a 400 kN capacity universal testing machine at a load rate of 3 mm/min. The load was applied at two points with a load span of 300 mm. Figure 1 shows the test set up for the flexural behaviour. To minimise the indentation failure, steel plates were added at the support and loading points. The strain on the top and bottom faces of the beams were measured by using four uniaxial strain gauges (types PFL-20-11-1L-120). The load was increased until failure. The load and mid-span displacement were measured with the electronic load cell and a laser displacement transducer connected to a Data Acquisition System, respectively.

TEST RESULTS AND DISCUSSION

Moment-deflection behaviour

Figure 2 shows the moment-deflection response of the tested beams. The hollow beams (BH) showed a liner elastic behaviour until failure at 31 kN.m. The average ultimate flexural stress at the top and bottom of the tested beams was 291 MPa. This stress value is approximately 45% of the compression failure stress based on coupon test results for GFRP profile. The main reason behind this behaviour is compression local buckling of the top flange at the constant moment region. The buckling is then followed by separation of the web-flange junction, delamination and crushing of the web. On the other hand, figure 2 shows the significant gain in strength and stiffness of the filled beams compared with the hollow beams. The overall behaviour of the filled beams is linear elastic until failure. The average failure moment of the filled beams BC-10, BC-37 and BC-43 is greater than the hollow beams by 100%, 140% and 145%, respectively. It is due to the contribution of the concrete core which restricting and delaying the local buckling of the GFRP tubes, the filled beams capacity is higher. However, with 335% increase in concrete strength from 10 to 43 MPa, only 19% is the improvement of the strength of the section. Two possible reasons might be causing this behaviour. Firstly, the brittleness of the concrete is increased with the increasing concrete strength. Secondly, the overall behaviour of the filled beams is controlled by the behaviour of the outside tube.

![Figure 1 Experimental set up details](image1.png)

![Figure 2 Moment-deflection behaviour of the tested beams.](image2.png)
Figure 3 shows the flexural stiffness of the tested beams. The average flexural stiffness, $EI$ of the hollow beam is $2.38 \times 10^{11}$ N:mm$^2$. This value was calculated using the following equation:

$$EI = \frac{P a}{48 \Delta} (3L^2 - 4a^2)$$

(1)

Where $EI$ is the effective flexural stiffness in N:mm$^2$; $P$ is the applied load in N; $a$ is the shear span in mm; $\Delta$ is the mid-span deflection mm; and $L$ is the span in mm. The calculated un-cracked flexural stiffness of the 10 MPa and 43 MPa concrete are $3.19 \times 10^{11}$ N:mm$^2$ and $4.19 \times 10^{11}$ N:mm$^2$, respectively. These values are 34% and 75% higher than the stiffness of the hollow beams. These results reflect the contribution of the concrete core in the overall stiffness of the filled beams. On the other hand, the flexural stiffness of the beams after the initiation of the tensile cracks in the concrete is approximately 26% higher than the hollow beams for all types of concrete. These results might be attributed to the difference in the cracking stress between the 10 MPa and 43 MPa concrete that results in lower contribution of concrete area for higher concrete strength than lower concrete strength to achieve the equilibrium of the internal forces.

Figure 3 Flexural stiffness of un-cracked and cracked sections.

**Moment-strain relationship**

The moment-strain relationships of the hollow and concrete filled beams are shown in Figure 4. The figure shows that the hollow section failed at compressive strain of 3140 microstrains and tensile strain of 6100 microstrains. The compressive strain tends to become positive which reflecting the local buckling effect on the top flange as shown in Figure 4a. In contrast, the filled beams failed at tensile strain of 12400, 14800 and 14820 microstrains for BC-10, BC-37 and BC-43, respectively as shown in Figure 4b. These strain levels are much higher than that of hollow beams in addition it approximately represents 77, 92 and 93% of the failure strain of the pultruded GFRP section based on coupon tests (Table 1). These results indicate that no tension failure had occurred at the tension site. Similarly, the tested beams showed a higher level of compressive strains of 11100, 11200 and 11250 microstrains for BC-10, BC-37 and BC-43, respectively. Again, these values are much higher than the strain levels of hollow beams and it approximately represents 97, 98 and 98.5% of the ultimate compressive strains of GFRP section. The results indicate that the filled beams failed at onset of the compression failure.

Figure 4 Moment-strain relationship of hollow and filled beams.
Modes of failure

The failure of the hollow beams started with the local buckling of the top flange followed by separation of the web-flange junctions and by delamination and crushing in the web as shown in Figure 5a. Similar behaviour has been reported in the literature (Guades, Aravinthan & Islam 2014; Vincent & Ozbakkaloglu 2013). On the other hand, the failure of the filled beams began with an inflated flange at the compression side due to the high compressive strains followed by cracks in the fibres in the transverse direction. As the concrete core cracked, the failure progressed into the side as shown in Figure 5b, c and d. The complete failure occurred after the fibre cracking in the top flange and the webs of the section at a strain level of approximately 11200 microstrains. The strain level is far greater than the failure strain of the hollow beams which illustrates that the concrete infill prevented the occurrence of the local buckling. The results also indicate that no failure has been occurred in the tension side due to the fact that the tensile strain levels are still lower than the ultimate level.

CONCLUSIONS

This paper presented the experimental results of four point bending tests on concrete filled GFRP tubes (CFFTs). Concrete compressive strength was the main parameter examined in this research. Based on the results, the following main conclusions can be addressed

- The concrete filled GFRP sections show a considerable strength and stiffness increase with 100-145% and 26%, respectively compared with the hollow sections.
- Increasing the compressive strength of the concrete core from 10 MPa to 43 MPa result in increasing the ultimate moment by 19%. However, the flexural stiffness of the filled beams is almost the same.
- In view of using concrete filled GFRP tubes as a viable structural solution to prevent local buckling of the tube, low strength concrete can be considered as a practical infill for the GFRP tubes.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the support of Wagner’s Composite Fibre Technologies (WCFT) for providing the GFRP pultruded sections. The first author would like to acknowledge the financial support by the Ministry of Higher Education and Scientific Research-Iraq.
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DEVELOPMENT OF A SMART GFRP HONEYCOMB SANDWICH FOR INTERACTIVE BRIDGES

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ABSTRACT

Nowadays free formed buildings with organic shapes and biomorphic structures in combination with resource and energy efficiency as well as functional integration are getting more and more in the focus of modern civil engineering. The Basis of such innovative buildings are conventional construction materials like steel, glass and reinforced concrete, whereby these classic materials are limited regarding requirements as degree of lightweight, freedom of design and efficiency as wells as functional integration. For this reason, innovative buildings often cannot be realised or only with high costs and expenditure of resources. Within the scope of the research project “New lightweight structural components and processing technologies for the application in support structures”, supported by the Sächsische Aufbaubank SAB, the theoretical and experimental basis for a new function integrated, modular support structure in lightweight design was established. The focus here was on the designs development of a support structure with integrated sensor technology for bridge monitoring and lighting control and its realization by the use of new production technologies. Especially the use of glass-fibre reinforced plastics (GFRP), because of their low weight, excellent mechanical properties and corrosion resistance as well as the possibility of a load-adjusted dimensioning, resulted in new solution approaches. In connection with the realization of the production, different technological concepts were analysed with reference to their suitability, integration of required force transmission and further functions during and after production. The analysis of the lightweight elements with regard to their production and their mechanical properties resulted in a holistic production and tool concept, which represents the whole process chain from the textile to the structural component. The transfer of the results into practice was done within the construction of a new smart GFRP honeycomb arch bridge with interactive lighting in Chemnitz (Saxony).

KEYWORDS

lightweight construction, GFRP, honeycomb sandwich structures, interactive, functional integration, arch bridge

INTRODUCTION

The focus of modern architecture is increasingly on free-formed buildings with organic shapes. Design and form-finding process are therefore increasingly conducted via complex, computer-assisted methods. The implementation of these form-finding process is largely dependent on the performance of the material. This is also challenging. For the construction and operation of buildings, creative aspects are as important as demands for load capacity, function implementation, resource and energy efficiency. These demands cannot be realized with conventional building materials alone. New approaches are required, which include the development of innovative bond materials by combining hardening, functional and shaping material structures (Ehrlich and Gelbrich 2014). Inspired by properties of natural lightweight structures, like marine algae and plant leaves, bionic approaches and natural structures are increasingly adapted for technical applications (Gelbrich and Pfalz 2010). In modern architecture as well as in nature, efficient lightweight design is essential for bridging maximum spans (Ehrlich and Gelbrich 2014). The focus is on a material-saving, weight-reducing way of construction, with light designs based on nature. This is to be achieved by replacing heavy material with light bond material.

The implementation of lightweight design thus involves lightweight material in combination with support structures that are dimensioned in a load-adjusted way. Efficient, functionally integrated material combinations are needed. Further research is necessary, particularly in the field of fiber-reinforced composite material for the application in civil engineering (Ehrlich et al. 2011).
FROM THE FORM-FINDING-PROCESS TO MANUFACTURING

Form-finding-process and selection of material

The main challenge in the development of support structures in lightweight design for resource-efficient bridges was to realize maximum spans with a minimum of weight in structure and with materials of low density and with integrated functions like monitoring and lighting. Based on these key points, the design was conducted, the material selected and finally a reference object built.

The starting point of the form-finding-process was the workshop “Form-Finding: form follows material”, that accompanied the research project. In this workshop, architects worked out a lot of creative ideas and designs for innovative lightweight bridges. Particularly, the bridge design of the architectural firm Beier.Steiner Architekten und Ingenieure was selected to execute a closer examination of honeycomb support structures in bridge designs (Figure 1).

GFRP was selected as material for the bridge modules. In combination with the structure, chosen for the bridge design, this led to a support structure with low use of materials and low dead load. Translucent GFRP covers were used as filling material for the honeycombs. By the application of translucent GFRP covers in combination with integrated LED technology, different lighting scenarios can be realized.

GFRP has many advantages for building complete support structures, given that design and production are appropriate for the material. GFRP is not only translucent, it is also resistant to environmental influences and many aggressive substances, such as de-icing salt. GFRP combines very good mechanical properties and low weight. When designed and produced in a load-specific way, it ages only to a small degree. Examinations of dynamically loaded laminates over a period of 30 years showed no negative changes in the supporting laminates [Hönninger et al. 1999]. Thus, extremely durable buildings can be produced that need only low maintenance.

Construction, design and optimization

Based on the idea to use a honeycomb as basic form for the innovative support structure in bridge elements, a honeycomb-sandwich (Figure 2) was implemented into a feasible solution, appropriate to material and production. Inspired by lightweight constructions in nature, the honeycomb-sandwich structure was developed following the natural honeycomb, which consists of evenly shaped hexagonal cells and has an ideal ratio between wall material and volume. Due to its stiffening function, it is often used in engineering to stabilize constructions. With an optimal material input, honeycombs ensure a uniform distribution of force within the component and thus meet the requirements imposed on lightweight design.

The sandwich design was chosen for the implementation of the innovative support structure. Components of this design are characterized by a high second area moment and low material input. This is particularly favorable for components exposed to great bending strain. The honeycomb core acts as basic material for transmitting shear forces and increasing the space between the covering layers, which are exposed to tension and pressure. Both basic material and covering layers consist of GFRP (Figure 2).

The finite elements method (FEM) is used to design and optimize the GFRP honeycomb module. The geometry model of the GFRP honeycomb module was conducted fully parametrically, to permit a fast adaptation of the geometrical situation. In this way, different variant types could be calculated. The geometry (length, width, height) of the single honeycombs can be modified via the corresponding parameters, their number can be modified via length and width of the GFRP honeycomb module. The module is furthermore divided into the four elements upper...
and lower covering layer, honeycomb core and module border. The thickness can be determined for each element separately.

![GFRP honeycomb module in sandwich design](image)

Figure 2 GFRP honeycomb module in sandwich design

The discretization was realized with shell elements, to keep the computing time for the simulation within reasonable limits. The computing time for a single run is of considerable importance, especially with regard to the subsequent optimization, since many runs are necessary for the optimization to get reliable results.

Apart from the dead load of the GFRP honeycomb module, a distributed load and a single load in the middle of the honeycomb are forces that act on the GFRP module in the preliminary design. The bearing is realized by the two edges upon which the GFRP module rests during the mounting process. The maximum principal and shear stress and the maximum deflection are analyzed for the evaluation of the GFRP module, in accordance with the ultimate limit state and the serviceability limit state (Figure 3).

![Framework conditions and results of the simulation](image)

Figure 3 Framework conditions and results of the simulation

The optimization of the GFRP module is conducted by means of an implemented calculation algorithm which is integrated into the software. The parameters that are going to be optimized have to be activated, the framework conditions (external influences, storage), voltage and deformations limits have to be defined. From a certain number of calculation runs results a GFRP honeycomb module with a set-up that is perfectly adapted to the defined conditions.

![Testing and simulation of a single honeycomb](image)

Figure 4 Testing and simulation of a single honeycomb

To verify the simulation, the results were compared to the values measured in pressure tests of single honeycombs (Figure 4). In those tests, the single honeycombs were loaded with a compression force of 10 kN on an area of 10...
cm x 10 cm in the middle of the honeycomb. Tests and simulation showed a very good agreement of the shift values. With a deviation of 6 %, the simulation is very suitable for the preliminary design of the GFRP honeycomb modules.

Based on the simulation results of the preliminary design, the final structural dimensioning was conducted with the software Sofistik. Sofistik provides simple tools to display and calculate a variety of load combinations. However, the parameterization is extremely time-consuming, which justifies the splitting of the design process into preliminary design with ANSYS Workbench and final structural dimensioning with Sofistik.

**Implementation of production**

Two different technology concepts were developed and tested to implement the production of the structural elements. The first concept includes the production of the single honeycombs using the winding method, followed by agglutinating the single honeycombs to honeycomb modules (Figure 5). The winding method was chosen with regard to the dimensions of the components, the necessary reinforcing brackets and the notified medium-sized batches. The mounting of the single honeycombs requires small tolerances within the single honeycombs to keep addition errors and significant deviations within the module at a low level.

![Figure 5 Process chain from toolmaking to complete single honeycombs](image)

Producing the modules in the winding method is not feasible for curved bridges, because it is too much effort to design and produce the different winding cores.

The second concept includes the production of geometrically adjusted half-honeycombs in the wet-in-wet technology. The honeycomb core is made out of joint laminate strips in a meander-like shape with a thickness of 8 mm, which are stuck together and pressed into the wet-laminated cover layer afterwards (Figure 6). Due to their design, the laminate strips that are used for the production of the cores facilitate the production of different module types. The costs for tools are low, compared to the production of the modules from single honeycombs, because fewer molds are needed to depict the curved modules.

For the pre-assembly of the honeycomb-core a mounting aid was developed to facilitate the agglutinating of the laminate strips (Figure 6). The mounting aid consists of a 60 mm thick multiplex board, into which the honeycomb pattern was cut with a CNC cutter to guarantee the required tolerances.

![Figure 6 Production of the modules from meander-formed laminate strips with slip-resistant wearing layer](image)

A further step in the production of the modules was the application of a slip-resistant wearing layer onto the cover layer, which is the walking layer of the lightweight honeycomb bridge. Different types of quartz sand with differently sized grains were tested during the production of the single honeycombs to guarantee the required slip-resistance.

**Functional integration**

In extensive research work over recent years, various structurally integrated sensor systems have been developed to monitor fibre-reinforced plastic components. The Department of Lightweight Structures and Polymer
Technology at Chemnitz University of Technology in cooperation with the Kompetenzzentrum Strukturleichtbau e.V., Chemnitz, designed textile-based stitched sensors and tested these in various applications (Ulbricht et al. 2011). To control the desired bridge lighting, the integrated stitched sensor system has been specially adapted to meet the particular requirement of interactive walking. The task of the integrated sensor system is to detect the position of pedestrians on the GFRP bridge elements and then control the switch function for the automatic bridge lighting.

The sensor antennas consist of insulated copper strands and are connected segmentally via a cascaded signal-processing unit (Figure 7). Combined with the sensor system a consistent lighting of the translucent GFRP honeycomb modules can be reached by the use of LED lights.

The electronics creates a localized electromagnetically field on the antennas, which allows a feedback after a change of the electrical resistance and/or the dielectricity. When a pedestrian steps on a sensor integrated bridge segment, the electrical resistance and dielectricity changes due to the isolating shoe and food. The resulting field characteristic of the sensor causes a switching function in the electronics unit. The electronics unit sends a signal to the control of the bridge and switches on the lighting.

![Figure 7 Integrated sensor system for bridge monitoring and lighting control](image)

An innovative lightweight honeycomb bridge was developed and implemented in Chemnitz in the scope of a research project. It was built in the course of revitalizing and converting the former dyeing factory Haase into a company building and connects it with the opposite bank of the river Chemnitz. In accordance with the local

![Figure 8 Lightweight honeycomb-bridge in Chemnitz](image)
situation, especially flood protection, the bridge was realized as a footbridge with a span of 30 m and a pavement width of 2.50 m (Figure 8).

The shape and design of the reference object comprises a bridge with an unequal round arch. A steel support served as support structure, together with a spanning pipe turn. The pavement is mounted into the third points of the pavement construction, by means of two hanger pairs.

The pavement construction has a curved floor plan. It consists of torsion-free main girders lengthwise and functionally integrated lightweight honeycomb modules made of GFRP in transverse direction. The lightweight honeycomb modules have a width of 2.60 m and a height of 276 mm (upper covering layer: 17 mm, core: 250 mm, lower covering layer: 9 mm). The modular fibre composite construction between the main girders is a flat honeycomb structure that acts as secondary support structure, which spans the steel girders. The pavement is embedded on both sides of the river. Massive abutments are used for this purpose, consisting of textile concrete in modular composite design. A functionally integrated exposed-concrete formwork was developed and applied.

CONCLUSIONS AND OUTLOOK

As saving of resources, freedom in design and functional integration are becoming more and more important, the focus in architecture and civil engineering shifts to lightweight design. Without a significant increase of resource efficiency, future-oriented designs are hardly realizable. Materials for lightweight design have to be highly effective and multi-functional hybrids. Fiber composites are ideally suited to meet these requirements.

Innovative, functionally integrated support structures have been developed within the scope of a research project. They are produced in GFRP sandwich design with stiffening honeycomb cores. Thus, they are the basis for combining innovative lightweight technologies and progressive functional design in support structures. Due to the strict restrictions in civil engineering the GFRP honeycomb modules are to be used as secondary support structures in larger bridges with spans of more than 15 m. For smaller bridges with spans up to 15 m no steel supports are needed and support structures in GFRP sandwich design are feasible. These support structures can be prefabricated in the factory. Small cranes are sufficient for the on-site mounting, due to the low weight of the structure. This significantly increases the effectiveness of the mounting process.

Although fiber composites have been further developed in recent years, they were rarely used in civil engineering. Architects, planners and building authorities are lacking experience, there are no reliable parameters for durability, standards, guidelines, approvals and accepted test methods. Thus, there is no basis for the application of fiber composites in support structures.

The main objective of research and development now is to create a base for the application of fiber composites in support structures by developing innovative lightweight material composites, technologies for their production and integrating suitable sensors for monitoring. A close connection of research and practice (building companies, architects, and planners) and cooperation with approving authorities are decisive factors. A couple of reference objects were implemented successfully in the recent years. This is a decisive step towards the approval of new lightweight materials and composites.

ACKNOWLEDGEMENTS

The research project “New lightweight structural components and processing technologies for the application in support structures” was successfully realized thanks to the close and always fruitful cooperation between Fiber-Tech, Steelconcept, Hentschke-Bau, Stfi and TU Chemnitz. Great thanks also goes to the development fund provider Sächsische Aufbaubank SAB, DFG Deutsche Forschungsgemeinschaft, the engineering office Schulze&Rank, the architects Beier, Steiner Architekten and engineers Dankhard Remmler, ARC Architekturconcept and Architekturkanal for supporting the project.

REFERENCES


EXPERIMENTAL STUDY ON THE PERFORMANCE OF GFRP CONNECTORS FOR USE IN PRECAST CONCRETE SANDWICH PANELS

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ABSTRACT

In this paper, three kinds of glass fiber reinforced polymer (GFRP) connectors were proposed for use in precast concrete sandwich panels, including “I” beam, corrugated plate and hexagonal tube. Direct shear tests based on a typical push-out configuration (i.e., three concrete wythes linked with connectors and two rigid foam layers in between) were conducted to compare their performance with that of one commercially available product, which is being widely used in PCSPs in prefabricated construction. The test results indicated that, the shear resistance of all proposed connectors was governed by the material failure in GFRP and no pull out failure of the connectors from the concrete wythes occurred. All proposed GFRP connectors showed a higher shear stiffness and shear resistance (more than 13kN per connector) than corresponding existing systems. The corrugated GFRP plate performed the best while the GFRP tube performed the second in terms of shear stiffness and shear resistance, demonstrating their good potential for applications in partially or fully composite concrete sandwich panels. Finite element analyses were also conducted to investigate the stress condition in the proposed connectors during the loading stage. The analytical results indicated that “I” beam and corrugated plate connectors mainly generate shear deformation whereas the hexagonal tube generate both significant flexural and significant shear deformation.

KEYWORDS

Precast concrete sandwich panel, GFRP tube, connector, direct shear test, shear stiffness, shear resistance.

INTRODUCTION

Precast concrete sandwich panels (PCSPs) consist of inner and outer reinforced concrete (RC) or prestressed concrete (PC) wythes, a core foam insulation layer, and connectors penetrating through the insulation. They have been widely used in office and residential buildings particularly in Europe and North America to achieve both structural and thermal efficiency. According to the degree of the composite action, PCSPs can be divided into fully composite, partially composite and non-composite types (Edward 2005). The degree of composite action mainly depends on the stiffness and resistance of connectors that transfer in-plane shear force between two concrete wythes. Traditionally, solid concrete blocks and steel bent up bars (i.e., steel trusses) were used as the connectors in PCSPs (PCI 2011). Although PCSPs with these traditional connectors exhibit a higher degree of composite action, they lead to the thermal bridge effect due to the higher thermal conductivity of steel and concrete, and consequently the panel thermal efficiency is largely compromised. In order to solve this problem, fiber reinforced polymer (FRP) composites have been introduced to manufacture the connectors in various forms owning to their lower thermal conductivity and high strength (Salmon et al. 1997). Currently, the widely used FRP connectors include pin-type glass fiber-reinforced polymer (GFRP) (Woltman et al. 2013), grid-type carbon FRP (CFRP) and GFRP (Gleich 2007; Frankl et al. 2011; Kazem et al. 2015), truss-type GFRP (Choi et al. 2015) and plate-type GFRP (Pantelides et al. 2008; Chen et al. 2015; Lameiras et al. 2013). Among them, the plate type and grid type connectors are often used to manufacture the composite type PCSPs because of their relatively high shear stiffness and shear resistance. However, both the plate and the grid types of connectors have significantly different stiffness in two orthogonal directions and usually can only designed to transfer one-way shear force. In addition, the issue of how the section of FRP connectors influences the shear transfer efficiency remains unknown.

This paper aims to propose optimal one-way and two-way FRP connectors and to evaluate their performance. Three types of new GFRP connectors were proposed, including “I” beam shape, corrugated plate shape and hexagonal tube shape. In order to evaluate their shear performance, direct shear tests were conducted through the widely used push-out configuration. Besides, shear performance of a typical commercial GFRP connector, which has been widely used in PCSPs in prefabricated building industry was also examined for comparison. The failure
modes, shear stiffness and shear resistance of the above-mentioned four kinds of connectors were observed and discussed, providing valuable information for further optimization of these FRP connectors.

PROPOSED FRP CONNECTORS

The three kinds of GFRP connectors and their connection details in PCSPs are shown in Figs.1(a)–1(c). The “I” beam and corrugated plate connectors were mainly proposed for one-way PCSPs in which the connectors only need to transfer the in-plane shear force along the longitudinal direction. The “I” beam FRP connectors belong to the plate type but provide more reliable anchorage to the two concrete wythes. It should be noted that the out-of-plane buckling often occurs in plate type connectors. To delay or avoid the buckling, corrugated plate type GFRP connectors was proposed (Figure 1(b)). The hexagonal GFRP tubular connector was mainly proposed for PCSPs, in which the connectors need to transfer two-way in-plane shear (e.g., Benayoune et al. 2008). Compared to solid sections, the tubular section is thought to be optimal because it provides more transverse stiffness when the connectors are subjected to flexure/shear in PCSPs. It should be mentioned that the GFRP connectors bear both shear and flexure in PCSPs although they are called “shear connector” in literature (e.g. PCI 2011). Considering the manufacturing convenience and structural efficiency, GFRP tubes with a hexagonal section was first explored in this paper.

All proposed GFRP connectors were made of resin impregnated dry glass fiber sheets through vacuum-assisted wet lay-up method. The glass fiber sheets used were biaxial type provided by Colan Australia Inc. and the areal weight is 855g/m². The epoxy resin used was provided by West System Inc. At first, the mechanical properties of GFRP formed based on the above fabric and resin were tested. The tensile modulus and strength of resin impregnated glass fiber sheets were evaluated through uniaxial tensile test of GFRP coupons with a fiber orientation of 0° & 90° (ASTM D3039) while the shear modulus and strength were evaluated through uniaxial tension test of GFRP coupons with a fiber orientation of ±45° (ASTM D3518). The coupon test set up can be found in Figure 2. The test results are presented in Table 1.

<table>
<thead>
<tr>
<th></th>
<th>Tensile modulus (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>Shear modulus (MPa)</th>
<th>Shear strength (MPa)</th>
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<td>GFRP coupon</td>
<td>32090.7</td>
<td>580.2</td>
<td>2925.6</td>
<td>21.0</td>
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</table>

The GFRP laminates in all proposed connectors were composed of four plies of fiber sheets with fiber orientation of ±45°. The thickness of the formed laminates varied between 2.4mm and 2.7mm. The overall heights of the connectors were 160mm for the “I” beam shape, 166mm for the corrugated plate and hexagonal tube shape, respectively. For the “I” beam connector, both the length and the width of the flanged part were 110mm (Figure 2).
3(a)). For corrugated plate connector, the overall length and each wavelength were 160mm and 40mm, respectively (Figure 3(b)). For hexagonal tube connector, the side length of the cross sectional hexagon was 40mm (Figure 3(c)). Figure 3(d) is a commercial FRP connector that is being popularly used for the construction of partially composite PCSPs.

IN-PLANE DIRECT SHEAR TESTS

A total of 8 specimens were tested for the above-mentioned four types of connectors under direct in-plane shear. Two identical specimens were fabricated for each type. The specimen IDs are indicated in Table 2. The “I” beam, the corrugated plate, the hexagonal tube and the commercial connector are symbolized as “I”, “T”, “H” and “C”, respectively. The direct shear tests were conducted using the popularly used push-out configuration (e.g., Tomlinson et al. 2016). The overall dimensions of the test specimens were 400mm×300mm×300mm (length×width×height), representing two sandwich panels back to back. Details of the specimen geometry can be found in Figure 4(a). In the specimens, extruded polystyrene (XPS) foam with smooth surface condition was used as the insulation layer. The effect of insulation on the test result can be ignored due to its weak bond with concrete (Hassan and Rizkalla 2010). In order to measure the relative slip between the core concrete wythes and the two outer concrete wythes, linear variable differential transformers (LVDTs) were placed at the front and back of core concrete wythes as shown in Figure 4(b). The load was applied in a displacement-controlled manner at a loading rate of 1 mm/min. A load cell was placed at the top center of core concrete wythes to measure the load on force.
TEST RESULTS AND DISCUSSION

The direct shear test results are summarized in Table 2 and the shear force vs. relative slip relationships for each type of connector are shown in Figs. 5(a) ~5(d). The average response of two identical specimens of four types of connectors are compared in Figure 6.

<table>
<thead>
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<th>Connector type</th>
<th>( f_c ) (MPa)</th>
<th>( P ) (kN)</th>
<th>( P_a ) (kN)</th>
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<tr>
<td>I-1</td>
<td>“I” beam</td>
<td>35.0</td>
<td>11.2</td>
<td>13.2</td>
</tr>
<tr>
<td>I-2</td>
<td>“I” beam</td>
<td>35.8</td>
<td>15.1</td>
<td>13.2</td>
</tr>
<tr>
<td>T-1</td>
<td>Corrugated plate</td>
<td>35.0</td>
<td>33.5</td>
<td>35.1</td>
</tr>
<tr>
<td>T-2</td>
<td>Corrugated plate</td>
<td>35.8</td>
<td>36.6</td>
<td>35.1</td>
</tr>
<tr>
<td>H-1</td>
<td>Hexagonal tube</td>
<td>35.0</td>
<td>28.3</td>
<td>26.7</td>
</tr>
<tr>
<td>H-2</td>
<td>Hexagonal tube</td>
<td>35.8</td>
<td>25.1</td>
<td>26.7</td>
</tr>
<tr>
<td>C-1</td>
<td>Commercial plate</td>
<td>40.2</td>
<td>16.5</td>
<td>15.8</td>
</tr>
<tr>
<td>C-2</td>
<td>Commercial plate</td>
<td>41.2</td>
<td>15.2</td>
<td>15.8</td>
</tr>
</tbody>
</table>

where \( f_c \) is the cylinder compressive strength of concrete, \( P \) is the shear resistance of connector; \( P_a \) is the average shear resistance of two tested connectors.

From Figs.5(a)~(c), it seems that the shear force vs. relative slip relationships of proposed FRP connectors can be divided into three stages. Initially, the transferred shear force increased linearly with the relative slip between two
concrete wythes. When the shear force reached the peak value (around 24kN, 69kN and 49kN for I series, T series and H series specimens, respectively), the transverse shear failure occurred in the GFRP laminate (Figs.7(a)–7(c)). After that, the I and H series specimens exhibited a yielding plateau with the increase of relative slip. However, the T series specimens exhibited a sudden drop of shear force and followed by a stable plastic stage. Finally, fibers at the tensile region ruptured gradually and the transverse shear force decreased until the connector was completely broken. The slip between the insulation and the concrete wythes could be clearly observed in this stage (Figure 8(a)). For specimens with the commercial connector, the failure mode was similar as shown in Figure 7(d). All proposed FRP connectors could provide a shear resistance of more than 13kN and their transverse stiffness are much higher than that of the commercial one, indicating that they have good potential for applications in partially or fully composite PCSPs.

There was no shear failure observed in XPS insulation. The surface of XPS insulation remained intact after tests as shown in Figure 8(b). Due to the smooth surface of XPS insulation board, it can be assumed to contribute no shear stiffness and shear resistance. The obtained shear force vs. relative slip curves reflected fully the performance of FRP connectors.

FINITE ELEMENT ANALYSIS

A preliminary finite element (FE) analysis has been conducted to investigate the stress condition in the proposed connectors during loading (for the linear stage only). Herein, general FE program ANSYS was used. Eight nodes solid element (SOLID45) was used for concrete wythes. The elastic modulus and Possion’s ratio were gave the values of 30GPa and 0.2, respectively. Multi-layer shell element (SHELL181) with 8 layers was used to model the GFRP laminate, as shown in Figure 9(c). In each layer, the GFRP material was treated as a linear elastic orthotropic
material. The elastic modulus along the fiber direction \(E_{11}\) and the shear modulus (assumed \(G_{12}=G_{13}=G_{23}\) according to Arman et al. 2006) used were the tested values as indicated in Table 1. The elastic modulus perpendicular to the fiber direction \((E_{22} \text{ and } E_{33})\) was equal to the resin’s tensile modulus provided by the manufacturer (West System Inc. 2016). For Poisson’s ratio \((\nu_{12}, \nu_{13} \text{ and } \nu_{23})\), a value of 0.3 was adopted for \(\nu_{12} \text{ and } \nu_{13}\). A value of 0.5 is adopted for \(\nu_{23}\) (Kishore et al. 2009).

Due to the symmetry, only half model was analyzed. The finite element models are shown in Figure 9, in which the connector and concrete elements shared the same nodes at contacted points. Nodes at the bottom of outer concrete wythes were restrained in all translational degrees of freedom. Vertical degrees of freedom of nodes at the top surface of core concrete wythes were coupled to ensure all nodes to move together. Load was applied in a displacement control manner. In the FE model, geometrical imperfection related to the first buckling mode of the connector was introduced. The amplitude of initial imperfection was set as 0.1 times of the GFRP laminate thickness (Debski et al. 2013).

After the analysis was completed, the vector plot of the 1st and 3rd principle strains was obtained and shown in Figs.10(a)~10(c). It can be seen that for both “I” beam and corrugated plate (Figs.10(a) and 10(b)), the principle strains are mainly along the orientation of approximate ±45°. However, for the hexagonal tube (Figure 10(c)), only the strain in “A region” is along the ±45° orientation. The tensile and compressive zones induced by tube bending can be observed in Figure 10(c). Therefore, the relative slip in cases of “I” beam and corrugated plate connectors was mainly composed of shear deformation, whereas in case of hexagonal tube it was composed of both significant flexural and significant shear deformation.
CONCLUSIONS

In this paper, three kinds of GFRP connectors have been proposed for use in PCSPs. Direct shear tests were conducted to evaluate their shear performance in comparison with that of a commercial FRP connector, in terms of the shear stiffness, shear resistance, failure mode and the full-range shear load-slip curve. The following conclusions have been arrived:

(1) All proposed connectors showed a linear behavior till the initiation of transverse shear failure in GFRP laminate, which was found to be progressive afterwards.

(2) All proposed connectors presented a considerable shear resistance of more than 13kN. Their shear stiffness was much higher than that of the used commercial one, indicating their good potential for use in partially or even fully composite sandwich panels.

(3) The preliminary FE analysis revealed that the deformation of “I” beam and corrugated plate connectors was mainly induced by shear whereas for the deformation of hexagonal tube connector both flexural and shear components were significant.

ACKNOWLEDGMENTS

The authors are grateful for the financial support received from Construction Industry Council, Hong Kong SAR (Project code: K-ZJK2) and the National Science Foundation of China (NSFC) Project Nos. 51278441. They are also grateful for a PhD studentship awarded to the first author by The Hong Kong Polytechnic University.

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EXPERIMENTAL INVESTIGATION OF FLEXURAL BEHAVIOUR OF THIN SANDWICH WALL PANELS REINFORCED WITH GFRP REBARS

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ABSTRACT

Pre-cast multi-layered concrete sandwich panels gain popularity in construction of commercial as well as residential buildings. Such panels consists usually of two concrete layers and an insulation layer in between. Both concrete layers, reinforced commonly with steel reinforcement, are joint together by use of connectors. Replacement of steel reinforcement by glass fibre reinforced polymers (GFRP) rebars enable the design of much thinner panels (40 mm for concrete layer) due to the reduction of the required concrete cover. A four-point flexural test was adopted to fully understand the load-carrying behaviour of sandwich panel components. The experimental study included instruments to measure the vertical deflection along the beam, strains inside of the connectors and shear deformation of insulation layer. Investigated parameters are the thickness of insulation layer, connectors’ arrangement as well as reinforcement ratio of the concrete wythes. This paper describes the results of experimental and theoretical investigation on flexurally stressed thin sandwich panels with internal GFRP reinforcement and connectors. Flexural load-bearing behaviour and transfer of shear stresses between the concrete layers are analysed.

KEYWORDS

Flexural tests, sandwich panels, connectors, FRP internal reinforcement.

MOTIVATION AND OBJECTIVES

Pre-cast multi-layered concrete sandwich panels usually consist of load-carrying concrete layer, insulation layer and facing concrete layer (see Figure 1). Currently they gain more and more popularity in industrial as well as residential construction industry. Therefore optimisation of their production and design process constitutes an attractive task as a direct response to the market needs. Application of glass fibre reinforced polymer (GFRP) rebars instead of traditionally used steel rebars enhances their thermal performance and due to much higher corrosion resistance of such rebars, in comparison to steel reinforcement, the investigated sandwich panels feature much thinner concrete wythes.

Figure 1 Components of sandwich wall panel

In case of commonly used sandwich panels, only one concrete wythe (the thick load-carrying concrete wythe) is considered to carry the whole bending load. The thinner facing concrete wythe acts solely as a protection against environmental exposure. For analysis of the investigated sandwich panels with double-sided thin concrete wythes, the contribution of both layers including the sandwich effect has to be considered. As it was proved in tests, even for panels with much lower stiffness of facing concrete wythe, the sandwich effect acting can provide significant load capacity reserves (Schmitt et al. 2014).
The objective of the experimental study is the investigation of the flexural load-carrying behaviour of complete sandwich panel system and its components.

**EXPERIMENTAL INVESTIGATIONS**

**Test series and test set-up**

The experimental programme including eight test specimens is presented in Figure 2. All specimens can be divided into four groups. In each group there are two specimens, with insulation thickness of 60 mm and 140 mm respectively, made of expanded polystyrene (EPS, compressive elastic modulus of 6 MPa). Thickness of both, upper and bottom concrete wythes of all specimens is 40 mm. Dimensions of test specimens are for all 1700x400 mm (LxW).

Varying parameters are:
1. thickness of the insulation layer
2. reinforcement ratio of concrete wythes
3. type of reinforcement (steel or GFRP)
4. arrangement of connectors

The GFRP rebars Schoeck ComBAR® (diameter of 12 mm) with elastic modulus of 60 GPa cut at both ends at the angle of 30° with length equal the thickness of the panel are applied as connectors between the concrete layers. The concrete mixture with maximal aggregate size of 8 mm was used. The compressive tests of 12 samples provided average cubic strength of 52.2 MPa with the standard deviation of 3.3 MPa. The test specimens of Group A are reinforced with usual steel rebars B500B with diameter of 8 mm and the same reinforcement ratio as Groups B and C. All the other sandwich panels are reinforced with GFRP rebars Schoeck ComBAR with a longitudinal elastic modulus of 60 GPa and diameter of 8 mm. Test specimens of Group D feature reinforcement ratio for each concrete wythe of \( \rho_D = 0.044 \), while in case of all other specimens reinforcement ratio is \( \rho_{A,B,C} = 0.031 \). In each panel the reinforcement of the upper layer is the same as of the bottom layer. Connectors’ grid for Group C is 20 x 30 cm while for Groups A, B and D is 20 x 50 cm. Test specimens of Group A reinforced with steel serve as a reference for GFRP reinforcement. Transversal reinforcement is applied in form of straight rebars of 8 mm with spacing of 240 mm.

<table>
<thead>
<tr>
<th>Group A - Steel</th>
<th>Group B - GFRP</th>
<th>Group C - GFRP</th>
<th>Group D - GFRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1 ( \rho_D = 0.031 )</td>
<td>S3 ( \rho_{B,C} = 0.031 )</td>
<td>S5 ( \rho_C = 0.031 )</td>
<td>S7 ( \rho_D = 0.044 )</td>
</tr>
<tr>
<td>S2</td>
<td>S4</td>
<td>S6</td>
<td>S8</td>
</tr>
<tr>
<td>500</td>
<td>500</td>
<td>300</td>
<td>500</td>
</tr>
</tbody>
</table>

All test sandwich panels were tested in four-point bending test. Load was recorded using 50 kN load cell. Vertical deflection along the test beam was recorded during the loading by use of five linear-differential digital transducers (see Figure 3). Flexural response of connectors was measured by strain gauges appliqued double-sided on the connectors close to the restraint in the bottom concrete layer. Relative displacement of concrete layers was measured at both ends of the test panels at the position of the most outer row of connectors using LVDTs.
Figure 3 Test set-up and measurement

Test procedure

Test specimens have been loaded displacement controlled with hydraulic cylinder with the loading rate of 0.5 mm/min until the cracking moment was captured. Then loading was applied in load increments with the loading rate 2 mm/min so that the crack pattern could be recorded and crack widths were measured (see Figure 4). The test panels were loaded up to the global failure or failure of a panel component considered as the end of service life of a structure.

Figure 4 Sandwich test panel S5 after shear failure of the insulation layer

RESULTS AND ANALYSIS

Flexural behaviour

In the analysis of the test results special attention is put to the behaviour in the range of load until reaching the service load. Generally, service load is defined as the ultimate load of the system divided by partial safety factor for material $\gamma_c$ and mean partial safety factor for action (dead and live load) $\gamma_F$ (Schmitt et al. 2014):

$$F_{\text{service load}} = \frac{F_u}{\gamma_c \cdot \gamma_F} = \frac{F_u}{1.5 \cdot (0.5 \cdot (1.35 + 1.5))} = \frac{F_u}{1.5 \cdot 1.425} = \frac{F_u}{2.14}$$

(1)

Resulting from the variety of materials and interfaces between them, number of failure modes can occur. Failure modes, which could be considered as eliminating the structure from further use (Ultimate Limit State), were: (1) shear cracking and bond failure of an insulation, (2) concrete crushing in a concrete wythe as well as (3) shear rupture of connectors. In all tests some or all of mentioned failure modes occurred up to the load around 20 kN. Using Eq. 1 the service load of 10 kN can be obtained. It need to be noted that in all tests global behaviour of the panels was very ductile and reaching ultimate load was indicated by large deformations. No abrupt failure occurred. Another failure modes can be recognized, describing the Serviceability Limit State: (1) excessive crack widths as well as (2) excessive deflections. Except those global failure modes, a number of failure modes of particular components of the system should be considered in order to create clear design recommendations.
Linear behaviour of panels was changing at the load ~5 kN (see Figure 5) when the first cracks occurred in the bottom concrete layer. From this point on, the flexural stiffness was decreasing along with development of the crack pattern. At the higher load levels (~15÷20 kN) insulation layer failed in shear or in bond to concrete or in combination thereof, causing significant drops of stiffness. For each test panel concrete crushing was the ultimate failure mode of the concrete layers, occurring at ~20÷25kN.

Mainly due to the lower modulus of elasticity of GFRP rebars, the developed crack widths were bigger than in case of panels S1 and S2 reinforced with steel. Nevertheless, at the service load of 10 kN maximal crack widths in the bottom wythe in any case did not exceeded 0.1 mm. Maximal observed crack widths bigger than 0.4 mm were reached at the very high load level while many other failure modes had already been developed. A uniform redistribution of crack pattern in both concrete layers was observed.

Influence of the parameters

Figure 6 and 7 show particular groups of specimens highlighted for comparison. Panels reinforced with steel (S1, S2) do not show higher stiffness in lower load range. It can be explained by the complicated distribution of internal forces along with development of damage in the insulation layer. Steel-reinforced panels, however, were stiffer in higher load range when the contribution of wythes’ stiffness itself had bigger influence on load-bearing behaviour. In a similar way, panels reinforced with higher reinforcement ratio (S7, S8) do not show any noticeable increase of stiffness. The highest stiffness and bearing capacity, also after development of cracks, was observed for test beams of Group C (S5, S6), which indicates great influence of amount of connectors and their arrangement on shear transfer capability. In each case, the panel with thicker insulation layer within each group showed lower stiffness, which can be contributed to relatively lower shear stiffness of insulation layer along with increase of its length.
**Load-carrying behaviour of the connectors**

Shear stiffness of a core layer (insulation + connectors) is essential for activation of sandwich carrying action. Although contribution of stiffness of polystyrene is significant in the lower load range, it decreases greatly after shear or bond failure of insulation. It was observed that insulation material fails at low load levels ~10÷20 kN and quality of its bond to concrete is highly variable, since the good adhesion of polystyrene to concrete is very dependent on the manufacturing process of sandwich panels.

In Figure 8 strains measured at both sides of the connector above the support (SG1,2) in test S5 are compared with the strains calculated from the measured relative displacement between the wythes (SRD1,2). Additionally in the figure the strain of connector in the second row (SG3,4) is depicted. In the calculation, the model of a bar restrained at both ends was adopted.

\[
\varepsilon = S_{RD1} = -1 \cdot S_{RD2} = \frac{6\mu l_c}{W_c d^3_A}
\]

where \(\varepsilon\) – strain of the connector derived from relative displacement of concrete panels, \(\mu\) - relative displacement of concrete panels, \(l_c, W_c\) - geometrical properties of connector’s cross section, moment of inertia and section modulus respectively, \(d_A\) – effective length of the connector (thickness of insulation layer + 2 * connector’s diameter) (ETAG-001).

Good agreement of calculation shows that stress in connectors can be derived directly from relative displacement of concrete layers with high accuracy.

**CONCLUSIONS**

In case of construction of thin concrete sandwich panels, the consideration of joint load-carrying action of both concrete wythes (sandwich effect) is essential to predict load capacity of the system. However, beside the global flexural failure a number of various failure modes can occur. Load-carrying behaviour needs to be in-depth considered, which is particularly important having the aim of preparation of clear guidance for the design of
sandwich panels. Relevance and impact on the global response of the structure of particular failure modes need to be analysed.

Test results showed that if the sandwich action is taken into account, the weakest point of the system is reliability and effectiveness of shear transfer between the concrete layers. Due to highly production-dependent bond properties of insulation, its contribution to the overall shear action, although substantial, need to be approached with conservativeness. The connectors remain the main shear transfer component in this system (after failure of the insulation), and therefore the deflection can be reduced by increase of the number of them.

Since the thin concrete layer cross-section is generally in over-reinforced condition (according to ACI 440.1R 15), the concrete crushing failure governs the design. Except smaller crack spacing and crack widths panels reinforced with steel rebars do not show remarkable difference in stiffness from panels with GFRP rebars within the service load range. Their ultimate load-bearing capacity, however, is higher by 20%.

Manufacturing of thin sandwich panels, except from complicated description of their load-bearing behaviour, is a challenging task, having in view practical implementation of the system into the construction market. Preparation process of test specimens proved that the providing of low concrete covers in case of thin concrete layers is possible under pre-cast plant conditions with satisfactory accuracy.

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DOUBLE-TUBE CONCRETE COLUMNS WITH AN FRP OUTER TUBE AND A HIGH-STRENGTH STEEL INNER TUBE

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ABSTRACT

Hybrid FRP-concrete-steel double-tube concrete columns (DTCCs) are a new type of hybrid columns proposed at The Hong Kong Polytechnic University. A hybrid DTCC consists of an outer tube made of FRP and an inner tube made of steel, with all the space inside the two tubes filled with concrete. This combination of materials is expected to lead to a very ductile, high-strength and corrosion-resistant column, even when both the steel and the concrete have very high strengths. In this paper, the results from a series of axial compression tests on hybrid DTCCs with a high-strength steel (HSS) inner tube are presented to demonstrate the excellent structural response of the new column form with a HSS steel tube. The test results confirm that in a DTCC with a HSS steel tube, the local buckling of the HSS tube is effectively prevented and the concrete is very well confined, leading to a very ductile column response. DTCCs thus provide an effective column form for the efficient use of HSS.

KEYWORDS

FRP, high-strength steel, concrete, tubular columns, hybrid columns, axial compression, confinement

INTRODUCTION

A significant number of studies (e.g. Rasmussen and Hancock 1995; Zhao et al. 2004; Ban et al. 2013) have been conducted on the use of high-strength steel (HSS) (defined herein as steel with a yield stress in the range of about 450 MPa to about 1000 MPa) products in civil engineering structures, but its use has so far been rather limited. One of the limiting factors for the structural use of HSS is the greater susceptibility of HSS members to buckling failures compared to normal-strength steel members. Such buckling failure means that the yield strength of HSS cannot be fully utilized and the ductility of the structural member can be greatly compromised.

Against the above background, a new form of columns incorporating a HSS tube has been recently proposed at The Hong Kong Polytechnic University (Teng and Yu 2015). The new column comprises two concentric circular tubes with all the space inside the tubes filled with normal-strength or high-strength concrete (Figure 1a). The outer tube, made of fibre-reinforced polymer (FRP), is primarily used to provide confinement and shear resistance to the column in addition to serving a protection skin for the column and as formwork during construction, so the fibres in the FRP tube are oriented close to the hoop direction. The inner tube, made of HSS,
provides the ductile longitudinal steel reinforcement needed by the column and additional confinement to the core concrete. These columns, referred to herein as double-tube concrete columns (DTCCs), have the following advantages over conventional concrete-encased HSS columns: (a) the FRP tube confines the column so that the strain capacity of concrete is greatly enhanced to ensure the full development of the yield strength and the ductile response of the HSS section under both tension and compression; (b) a steel tube is used instead of a typical open section, so that both tubes function as confining devices so the concrete is well confined; (c) the FRP tube provides additional protection to the steel section against corrosion; and (d) the two tubes eliminate completely the need for formwork and steel reinforcing bars. Indeed, the non-ductile responses of all three materials (for HSS this is due to buckling) in a DTCC are suppressed by their interaction, leading to a high-performance column that is highly resistant to both seismic loading and corrosion.

Hybrid FRP-concrete-steel DTCCs as a column form were first proposed as a variation that results from the filling of the inner void of hybrid double-skin tubular columns (DSTCs) (Teng et al. 2004, 2007) which consist of an outer FRP tube, an inner steel tube and a layer of concrete in between (Figure 1b). The concrete infill in the inner steel tube prevents inward buckling of the steel tube, which may occur in DSTCs when a strong outer FRP tube and/or a thin/HSS inner tube is used (Wong et al. 2008); it also significantly enhances the axial load capacity of the column. DTCCs are also an attractive alternative to DSTCs whenever the column size needs to be minimized; they are particularly attractive for use in high-rise buildings where the columns are normally subjected to a high axial load ratio and a reduced column size is attractive to achieve larger usable floor areas. The behaviour of DTCCs with a normal-strength steel tube has received some research attention (Ozbakkaloglu and Fanggi 2013, 2015; Zhang et al. 2015).

To demonstrate some of the expected advantages of DTCCs with a HSS steel tube, a series of stub column tests have recently been conducted. The results of these tests are presented and discussed in the present paper.

EXPERIMENTAL PROGRAMME

Test Specimens

Four hybrid DTCC specimens were prepared and tested, comprising two pairs of nominally identical specimens; the only difference between the two pairs was the thickness of the FRP tube. The specimens all had a nominal diameter (i.e. the outer diameter of concrete) of 300 mm and a height of 600 mm. The steel tubes inside the DTCC specimens all had an outer diameter of 260 mm and a thickness of 6 mm. The specimens were cast in two batches with the same concrete mix design for the two pairs of specimens, respectively. For ease of reference, each specimen is given a name, which starts with a letter “D” to represent DTCC specimens; this is then followed by a number (4 or 6) to represent the number of layers of fibres in the FRP tube. The Roman numeral at the end of specimen names is used to differentiate two nominally identical specimens.

When preparing the specimens, the FRP tube and the steel tube were used as the formwork for casting concrete. The two tubes for each specimen were levelled in a workshop and then fixed onto a wooden bottom plate. Strain gauges on the steel tube were attached before casting. Before testing, a 50 mm wide carbon FRP strip was applied at each end of the specimens to avoid premature failure in the end regions.

Material Properties

Two types of filament-wound glass FRP (GFRP) tubes were used in the tests. They had the same fibre winding angles of ±80 degrees to the longitudinal axis of the tube, and were formed from four and six layer of fibres, respectively; each layer had a nominal thickness of 0.42 mm based on the weight of fibres. The so-called curved coupon tensile tests, which were recently proposed by the authors’ group (Teng et al. 2016), were conducted to obtain the elastic moduli of the GFRP tubes in the hoop direction. In the curved coupon test, a curved coupon cut from a circular GFRP tube is directly pulled at the two ends; the instrumentation of the tests follows ASTM D3039/D3039M (2000). For each type of GFRP tubes, six curved coupons, each with a width of 35 mm and a gauge length of 150 mm, were tested. For the four-ply tubes, four additional curved coupons with a gauge length of 100 mm were tested to examine the effect of gauge length. The test results showed that the scatter of elastic moduli obtained from different coupons was small, and that within the tested range the gauge length had little effect on the results. The elastic modulus values obtained range from 35.6 GPa to 37.3 GPa except one tensile specimen which had a significantly larger value of 39.8 GPa and was excluded in calculating the average elastic modulus, which was found to be 36.3 GPa for all four FRP tubes.

Cold-formed HSS tubes were used in the DTCC specimens. Tensile tests on six steel coupons were conducted in accordance with BS18 (1987) for the HSS tubes used in the DTCC specimens. The coupons were cut from a
HSS tube along the longitudinal direction. The average values of elastic modulus and yield stress were found to be 191.9 GPa and 669.8 MPa respectively. In addition, two hollow steel tubes, which had the same height as those in the DTCC specimens (i.e. 600 mm), were tested under axial compression. Both steel tubes showed large plastic deformation until failure occurred by local buckling in the elephant’s foot mode. The average axial load capacity of the two steel tubes was found to be 3509.6 kN.

Three plain concrete cylinders (150 mm x 300 mm) were tested for each batch of concrete to determine the mechanical properties of concrete. The elastic modulus, compressive strength and compressive strain at peak stress of the concrete averaged from the cylinder tests were 28.9 GPa, 36.3 MPa and 0.00298 respectively for the D4 batch, and were 29.6 GPa, 37.2 MPa and 0.00298 respectively for the D6 batch.

Test Set-up and Instrumentation

For each specimen, four axial strain gauges and eight hoop strain gauges, all with a gauge length of 20 mm were installed on the outer surface of the GFRP tube; the same number of strain gauges with a gauge length of 5 mm were installed on the outer surface of the steel tube. The circumferential layout of the strain gauges is shown in Figure 2. In addition, for each specimen, four linear variable displacement transducers (LVDTs) were used to measure the overall axial shortening, while four other LVDTs were used to measure the axial deformation of the 250 mm mid-height region. All the compression tests were conducted using a 1000 tonne Universal Testing Machine with a displacement control rate of 0.5 mm per minute.
TEST RESULTS AND DISCUSSIONS

General Behaviour

All four specimens except Specimen D6-I were tested to failure, which was due to the hoop tensile rupture of the GFRP tube and was accompanied with loud noises. The test of Specimen D6-I was unexpectedly terminated when the load was close to the capacity of the testing machine. All four specimens displayed a continuous, monotonically ascending load-shortening curve. The four specimens after test are shown in Figure 3. The steel tubes of all the DTCC specimens were taken out for examination, and no significant buckling deformation was found, suggesting that the GFRP tube together with the concrete provided effective restraints to the inner steel tube against buckling deformation.

Axial Load-Strain Behaviour

The key results of the column specimens are summarized in Table 1. In this table, $P_u$ is the ultimate load of a DTCC specimen from the compression test, $\varepsilon_{cu}$ is the ultimate axial strain of the DTCC specimen, which is the strain at the rupture of the GFRP tube (for Specimens D4-I, D4-II and D6-II) or at the point when the test was terminated (for Specimen D6-I), $\varepsilon_{hu}$ is the ultimate hoop strain corresponding to the ultimate axial strain and was obtained by averaging the readings of the eight hoop strain gauges. In Table 1, the axial strain of unconfined concrete at peak stress, $\varepsilon_{co}$, found from tests on standard plain concrete cylinders is used to normalize the measured ultimate axial strain $\varepsilon_{cu}$. In this paper, compressive stresses and strains are defined to be positive, while tensile stresses and strains are defined to be negative, unless otherwise specified. It is evident from Table 1 that the ultimate axial strains of all DTCC specimens are very large, and up to around 10 times the peak axial strain of unconfined concrete.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Ultimate load $P_u$ (kN)</th>
<th>Ultimate axial strain $\varepsilon_{cu}$</th>
<th>Ultimate hoop strain of FRP tube $\varepsilon_{hu}$</th>
<th>$\varepsilon_{cu}/\varepsilon_{co}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>D4-I</td>
<td>8713.7</td>
<td>0.0240</td>
<td>-0.0174</td>
<td>8.27</td>
</tr>
<tr>
<td>D4-II</td>
<td>8445.1</td>
<td>0.0246</td>
<td>-0.0184</td>
<td>8.23</td>
</tr>
<tr>
<td>D6-I</td>
<td>9041.6</td>
<td>0.0254</td>
<td>-0.0167</td>
<td>8.52</td>
</tr>
<tr>
<td>D6-II</td>
<td>9420.8</td>
<td>0.0305</td>
<td>-0.0200</td>
<td>10.23</td>
</tr>
</tbody>
</table>

* Test unexpectedly terminated before failure

The axial load-strain curves of all four DTCC specimens are shown in Figure 4, where the axial strains were averaged from the readings of the four LVDTs covering the 250 mm mid-height region. Figure 4 shows that the DTCC specimens all had a bilinear axial load-strain curve with excellent ductility. The curves of Specimens D6-I and D6-II had a slightly steeper and longer second branch than those of Specimens D4-I and D4-II because of the use of a thicker GFRP tube in the former.

![Figure 4. Axial load-strain curves](image)

![Figure 5. Axial strain-hoop strain curves for the outer GFRP tube and the inner steel tube](image)
Lateral Dilation Behaviour

The axial strain-hoop strain curves of the outer GFRP tube for all the specimens are shown in Figure 5, where the hoop strains were averaged from the readings of the eight strain gauges installed on the GFRP tube while the axial strains were obtained from the average readings of the four LVDTs covering the 250 mm mid-height region. It is evident that the curves of the D6 pair are slightly higher than those of the D4 pair because of the use of a thicker GFRP tube in the former, but the differences between the two pairs of curves are very small. These differences are smaller than may be expected based on existing knowledge of GFRP-confined solid concrete columns, and are believed to be due to the existence of an inner steel tube, as further discussed later using a well-established analysis-oriented stress-strain model for FRP-confined concrete proposed by Jiang and Teng (2007).

Figure 6. Axial load-strain curves of DTCC specimens

Behaviour of Confined Concrete

In a DTCC specimen, the concrete is subjected to confinement from the inner steel tube in addition to that from the outer FRP tube, so its behaviour is expected to be different from that of concrete in an FRP-confined solid section column. Figure 6 is used to examine this issue. In Figure 6, the axial load-strain curves of the four DTCC specimens are each shown against three curves: the axial load-strain curve of the steel tube alone based on the hollow steel tube tests, the axial load-strain curve of the FRP-confined concrete (FCC) alone obtained using Jiang and Teng’s (2007) analysis-oriented model for FRP-confined concrete with the axial resistance of the FRP tube ignored, and that of the sum of the steel tube and Jiang and Teng’s (2007) predictions for the FRP-confined
concrete. As one of the two hollow steel tube tests was terminated at an axial strain of 0.021, the axial load-strain curve of the steel tube alone (averaged from the two hollow steel tube tests) beyond this strain value was extrapolated from the average experimental curve via an exponential extension for all the four DTCC specimens for comparison purposes. The extended portion of the axial load-strain curve of the steel tube alone for each DTCC specimen in Figure 6 is clearly indicated. In making the predictions for the FRP-confined concrete for each DTCC, the ultimate hoop strain obtained from the compression test of the DTCC specimen was used (see Table 1). It is evident from Figure 6 that the curves of DTCC specimens are much higher than the corresponding curves of the sum in the post-buckling stage of the hollow steel tubes, suggesting that the performance of DTCC specimens is far superior to what could be expected if there were no interaction between the steel tube and the concrete. The main benefit of interaction is the prevention of local buckling of the HSS tube by the FRP-confined concrete, demonstrating that DTCCs provide an effective column form for the efficient use of HSS. At failure as a result of the hoop tensile rupture of the GFRP tube, the load resisted by the DTCC exceeds the sum of steel tube and FRP-confined concrete by an average of 38.6% for the two D4 specimens and an average of 42.8% for the two D6 specimens. As the compression test of Specimen D6-I was unexpectedly terminated before failure, the load resisted by the DTCC specimen is compared in Figure 6 with the sum for the steel tube and the FRP confined concrete corresponding to the hoop strain in the GFRP tube at test termination.

It should be noted that in Figure 6, the curve of the steel tube alone was calculated using results from the hollow steel tube tests where the steel tube was subjected to uniaxial compression. In DTCCs, the steel tube is subjected to internal pressure due to the expansion of concrete inside the steel tube, as well as external pressure due to the existence of a GFRP confining tube. In general, the steel tube is expected to be in hoop tension and to confine the core concrete, which means that the internal pressure on the steel tube from the core concrete is larger than the external pressure on the steel tube from the FRP-confined concrete outside. In the four specimens tested in the present study, the inner steel tubes were all under hoop tension except during the very early stage of loading. As the steel tube in these DTCCs was under hoop tension, at the same axial strain, the axial load resisted by the steel tube is expected to be lower than that resisted by the same steel tube subjected to uniaxial compression only. This explains why the curve of the sum before the HSS tube buckles is generally higher than the corresponding test curve of DTCC (Figure 6).

The axial-hoop strain curves predicted by Jiang and Teng’s (2007) model for FRP-confined concrete in a solid circular section are also shown in Figure 5 for comparison. For these DTCCs, the existence of an inner steel tube did not affect significantly the axial-hoop strain relationship. The curve predicted by Jiang and Teng’s (2007) model is slightly lower than the experimental curves of Specimens D4-I and D4-II, suggesting that the steel tube may contribute more to constraining the lateral expansion of the column when the GFRP tube is relatively weak.

CONCLUSIONS

This paper has presented the details of a newly proposed form of FRP-concrete-steel hybrid columns, which is referred to as FRP-concrete-steel hybrid double-tube concrete columns (DTCCs). The new column consists of an FRP outer tube and a steel inner tube, with all the space inside the two tubes filled with concrete. The paper has also presented the results of the first set of axial compression tests on stub columns to demonstrate the favourable structural response of DTCCs. The results showed that the buckling of the HSS tube was effectively prevented and the concrete very well confined, leading to a very ductile column response. DTCCs thus provide an effective column form for the efficient use of HSS. In addition to resisting axial stresses, the HSS tube is also expected to provide confinement to the core concrete unless a very stiff FRP tube is used; the detailed mechanism of interaction between the FRP tube and the steel tube in DTCCs is a subject worthy of detailed analysis.

ACKNOWLEDGMENTS

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EXPERIMENTAL BEHAVIOUR OF HYBRID FRP-CONCRETE-STEEL DOUBLE-SKIN TUBULAR ARCHES

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ABSTRACT

Hybrid FRP-concrete-steel double-skin tubular members (DSTMs) are a recent development that exploits the advantages of FRP composites (e.g., corrosion resistance, high strength-to-weight ratios and tailorability of material properties) to achieve high-performance and durable structures. Hybrid DSTMs consist of an annular layer of concrete sandwiched between an outer FRP tube and an inner steel tube. This paper presents a preliminary experimental study designed to investigate the feasibility of using the DSTM cross-section in arches for use in bridge construction (i.e., double-skin tubular arches or DSTAs). The fibres of the outer FRP tube are oriented close to the hoop direction to provide confinement to the concrete annulus. The inner steel tube, while also providing confinement to the concrete annulus, is mainly used to provide the steel tensile reinforcement for the section to resist bending. The inner steel tube also acts as propping during the construction process while the outer FRP tube serves as the formwork during concrete casting, thus considerably simplifying the construction process and reducing the labour cost. The experimental programme presented in this paper consisted of three tests on hybrid DSTAs subjected to two closely-spaced point loads near the mid-span covering two different FRP tube thicknesses and two steel tube locations (i.e., concentric and eccentric steel tube positions). The experimental results are presented and interpreted to understand the behaviour of hybrid DSTAs, particularly how the FRP tube thickness and inner steel tube eccentricity affect the behaviour of hybrid DSTAs.

KEYWORDS

Double-skin tubular arch (DTSA), bridge construction, experiments, FRP tube, steel tube

INTRODUCTION

The use of fibre-reinforced polymer (FRP) composites in new construction, including the hybrid and optimal use of FRP with other materials (e.g., concrete, steel, timber) to create economical, corrosion-resistant structures, has become a topic of high interest in recent years (Holloway 2010). One of the most notable forms of FRP-based hybrid structural elements developed in recent years is the FRP-concrete-steel hybrid double-skin tubular member (DSTM) (Teng et al. 2007). These hybrid DSTMs consist of an outer tube made of FRP and an inner tube made of steel, with the space in between filled with concrete. The two tubes may be concentrically located (Figure 1a) to produce a section form more suitable for columns, or eccentrically located (Figure 1b) to produce a section form more suitable for beams (Teng et al. 2007; Wong et al. 2008). Hybrid DSTMs may be constructed in-situ or precast, with the two tubes serving as the stay-in-place form. The main advantages of hybrid DSTMs include: (a) excellent corrosion resistance, as the FRP tube is highly resistant to corrosion while the steel tube is protected by the FRP tube and concrete; (b) excellent ductility and energy absorption capacity, as the concrete is well confined by the two tubes and outward local buckling of the steel tube is constrained by the concrete; (c) a high strength/stiffness ratio as the inner void largely eliminates the redundant concrete; (d) ease of construction, as the two tubes function as a permanent form for casting concrete and the presence of the inner steel tube allows easy connection to other members (Teng et al. 2007).

Many studies have been carried out to understand the behaviour of hybrid double-skin tubular columns (DSTCs) (e.g., Teng et al. 2007; Yu 2007; Yu et al. 2010a,b; Fanggi and Ozbakkaloglu 2013; Idris and Ozbakkaloglu 2014; Ozbakkaloglu and Fanggi 2014; Zhang et al. 2015), leading to a sound understanding of the behaviour of DSTCs. In hybrid DSTCs, the FRP tube, with fibres oriented close to the hoop direction, offers mechanical resistance primarily in the hoop direction to confine the concrete and to enhance the shear resistance of the
member. Investigations into hybrid double-skin tubular beams (DSTBs) (e.g., Yu et al. 2006; Zhao et al. 2016) have revealed that DSTBs have a very ductile response under flexural loads; furthermore, an eccentric steel tube can significantly improve the flexural stiffness, ultimate load and cracking resistance of hybrid DSTBs (Yu et al. 2006).

![Cross-sections with a concentrically placed steel tube](image1)

(a) Cross-sections with a concentrically placed steel tube

![Cross-sections with an eccentrically placed steel tube](image2)

(b) Cross-sections with an eccentrically placed steel tube

Figure 1 Typical cross-sections of hybrid DSTMs

This paper presents the results of a preliminary experimental study designed to investigate the feasibility of using DSTMs as arches in bridge construction. The fibres of the outer FRP tube are oriented close to the hoop direction to provide confinement to the concrete annulus. The inner steel tube, while also providing confinement to the concrete annulus, is mainly used to provide the steel tension reinforcement for the section to resist bending. Hybrid double-skin tubular arches (DSTAs) not only possess a high strength-to-weight ratio, but also significantly reduce the construction cost of arch bridges due to the elimination of formwork.

**EXPERIMENTAL PROGRAMME**

**Specimen Design**

The curvature of the arch was selected to match an existing arch bridge, namely the Galena Creek Bridge in the United States (Durski 2010). Considering the physical laboratory and construction constraints associated with the size of specimens, only a segment of the arch was studied in the laboratory. The span of the test arches was selected to be 3600 mm and the height at crown was selected to be 480 mm (Figure 2a). In order to achieve a reasonable cross-sectional void ratio (defined as the ratio between the inner diameter and the outer diameter of the annular concrete section) with a reasonable concrete wall thickness, an inner diameter of 154 mm was selected for the FRP tube and a 88.8 x 4 circular hollow section (CHS) standard steel tube (which had an actual outer diameter of 90 mm) was adopted as the inner steel tube (Figure 2b). With these dimensions, a sectional void ratio of 0.58 was achieved. The diameter-to-thickness ratio of the steel tube was 22.2, which is within the limits (20-42) recommended by Yu (2007).

![Arch elevation](image3)

(a) Arch elevation

![Cross-sectional view of 1-1](image4)

(b) Cross-sectional view of 1-1

![Eccentricity](image5)

(c) Eccentricity

Figure 2 Arch geometry

In total, 3 arch specimens were designed and prepared in the laboratory. Instead of using prefabricated FRP tubes by filament winding, the FRP tubes for the arch specimens were formed in the laboratory via the wet-layup process by wrapping FRP layers on the concrete annular section. This alternative method of forming FRP tubes was expected to have little effect on the behaviour of the test arches. One of the three specimens was wrapped with 6 layers of glass FRP (GFRP) having a 1025 g/m² fibre density per layer, while the other two specimens had 4 layers of the same GFRP each. In all the arch specimens, the fibre direction was controlled to be between 0°-10° to the hoop direction of the tube. In one of the two specimens with 4 layers of GFRP, the steel tube was placed with an 18 mm eccentricity, while in the other specimens, the steel tube was concentrically located. The steel tube eccentricity is defined as the distance between the centre of the steel tube and the centre of the concrete annulus (Figure 2c). Further details of the specimens are summarized in Table 1.

688
A roller was used to squeeze out any excess epoxy. Next, a perforated steel tubes were bent to the required curvature using cold mandrel bending at a cold bending facility. In fifth and fourth, which epoxy and the geometry of the specimen, achieving a section of uniform thickness through the wet layup process. Due to the gravitational flow of excess adhesive. During the specimen construction process, a shrink tape was wrapped around and heated to apply pressure on the arch surface to form a GFRP tube. Once wrapped from the mould upon appropriate curing. These steel-concrete arches were next wrapped with the desired number of GFRP layers to form the external FRP tube.

Table 1 Details of DSTA specimens

<table>
<thead>
<tr>
<th>Batch</th>
<th>Specimen</th>
<th>FRP tube Density g/m²</th>
<th>FRP tube inner diameter (mm)</th>
<th>Eccentricity of steel tube (mm)</th>
<th>Steel tube</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>6P904C</td>
<td>6150</td>
<td>154</td>
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<td>90</td>
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<tr>
<td></td>
<td>4P904E</td>
<td>4100</td>
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<td>18</td>
<td>90</td>
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<td>2</td>
<td>4P904C</td>
<td></td>
<td></td>
<td>0</td>
<td>90</td>
</tr>
</tbody>
</table>

Each specimen has a unique six-character name as shown in Table 1. The first two characters indicate the number of FRP plies (i.e., “6P” means 6 plies while “4P” means 4 plies). The third and fourth characters represent the steel tube outer diameter. The fifth character indicates the thickness of the steel tube. The last character indicates the position of the steel tube within the concrete annulus, with “C” indicating a concentrically-placed steel tube and “E” indicating an eccentrically-placed steel tube. For example, specimen 6P904C was confined by a 6-ply FRP tube formed via the wet-layup process and had a concentrically-placed circular steel tube of 90 mm in diameter.

**Specimen Preparation**

In practical applications, the FRP tube should be prefabricated by filament winding and the steel tube should be placed inside the FRP tube before the casting of concrete, so that no temporary formwork is required for casting the concrete. However, prefabricating the FRP arch tube was found to be too expensive at this preliminary stage. Therefore, it was decided to first cast a concrete arch with a curved steel tube inside and then wrap the desired number of GFRP layers around the steel-concrete arch. The method involved the use of a mould, where the steel tube was properly positioned and restrained in the mould, for the casting of concrete to form a steel-concrete arch, which was then removed from the mould upon appropriate curing. These steel-concrete arches were next wrapped with the desired number of GFRP layers to form the external FRP tube.

A negative mould (i.e., the void for casting the concrete of a DSTA) was first produced in two halves, which were then joined to form a continuous negative mould. Each mould was designed to cast two arches simultaneously. The moulds were manufactured using high density foam and were then fixed to a backing plate (Figure 3a) to provide enough strength for carrying the weight of wet concrete. In order to water-seal the high density foam, a coating of PVA glue was applied to the exposed (i.e. internal) surface of the foam mould which was then followed by a layer of enamel paint. The two halves of the mould were fixed to a stiff steel frame to ensure that no relative movement between the two halves would occur. A photograph of the half moulds and the closed mould are shown in Figures 3a and 3b respectively.

The steel tubes were bent to the required curvature using cold mandrel bending at a cold bending facility. In order to ensure composite action between the concrete and the steel tube in a DSTA, shear studs were welded at a 100 mm spacing in the longitudinal direction at both the topmost and the bottommost points of the steel tube cross-section. The steel tube was then placed in the mould and the mould was set upright (Figure 3c). The steel tube was next fixed in place using an external mount welded to the mould frame (Figure 3c). This was necessary as the buoyancy acting on the steel tube during the casting of concrete applied lifting forces to the steel tube. Once all steps of the above procedure were completed, the concrete was poured (Figure 3c). Due to the limitations imposed by the existing mixer, three batches of concrete had to be mixed for the three DSTA specimens respectively. The same mix design was adopted for all three batches with a 28-day target cylinder compressive strength of 70 MPa. After 7 days of curing, the steel-concrete arch was removed from the mould.

After 28 days of curing, the steel-concrete arches were suspended vertically on a steel frame (Figure 3d). To avoid stress concentrations in the GFRP tube, jagged regions of the cast concrete tube were ground to a smooth surface. For specimen 6P904C, the glass fibre sheets were cut in to 100mm strips, impregnated with epoxy and then wrapped around the steel-concrete arch in a spiral with a 50mm overlap between passes until the end of the arch to form a GFRP tube. Once wrapped, a roller was used to squeeze out any excess epoxy. Next, a perforated release film was wrapped around the GFRP tube, and a peel ply and filter cloth were applied on top to absorb any excess adhesive. Finally, a shrink tape was wrapped around and heated to apply pressure on the arch surface and squeeze out any excess adhesive. During the specimen construction process, due to the gravitational flow of epoxy and the geometry of the specimen, achieving a section of uniform thickness through the wet lay-up...
process was found to be difficult. Therefore, GFRP prepreg sheets were used instead in the next two DSTA specimens to form the GFRP tube. The formation of a GFRP tube from the prepreg sheets followed the same process as used for forming the GFRP tube from fibre sheets for specimen 6P904C (Figure 3e). Once the GFRP tube was formed, a release film was applied, which was then followed by the wrapping of a shrink tape around the steel-concrete arch. Once all these steps were completed, heat blankets were wrapped around the arch to cure the adhesive. Specimens were cured for two hours at 120 degrees Celsius. A fully cured specimen is shown in Figure 3f. Once cured, the end regions of the arches were reinforced by 3 additional GFRP layers of 50 mm in width at each end.

![Negative mould halves](image1)
![Steel frame](image2)
![External mount](image3)
![Steel-concrete arch](image4)
![Wrapping of GFRP](image5)
![Completed specimen](image6)

**Figure 3 Construction of hybrid DSTAs**

**Material Properties**

Five tensile coupons each were made from a GFRP sheet and a GFRP prepreg sheet and tested for fibre direction tensile properties. All coupon tests were carried out according to the ASTM-D3039/3039M (2010) specifications. The average properties from the coupon tests are given in Table 2. The obvious differences in the elastic modulus between the two types of GFRP materials are believed to be due to a higher amount of adhesive in the GFRP sheets made using the wet lay-up process than the GFRP prepreg sheets. Three 100 mm x 200 mm concrete cylinders were made from each batch of concrete and tested for the 28-day compressive strength. The average properties from the concrete cylinder tests are also given in Table 2. The elastic modulus of steel was 195GPa while its yield stress was 270 MPa according to the manufacturer’s data sheet.

<table>
<thead>
<tr>
<th>Batch</th>
<th>Specimen</th>
<th>Concrete Compressive strength (MPa)</th>
<th>Concrete Elastic modulus (E_c) (GPa)</th>
<th>GFRP Elastic modulus (E_f) (GPa)</th>
<th>Ultimate stress (f_u) (MPa)</th>
<th>Tensile rupture strain (\varepsilon_f)</th>
</tr>
</thead>
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<td>487.0</td>
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</tr>
<tr>
<td></td>
<td>4P904E</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>4P904C</td>
<td>74.2</td>
<td>42.2</td>
<td>40.0</td>
<td>592.0</td>
<td>0.014</td>
</tr>
</tbody>
</table>

**Experimental Setup**

All three tests were conducted under two-point loading (with a 200 mm distance between the two loading points) using a 500 kN capacity actuator. An overall view of the experimental setup is given in Figure 4a. Specially designed bearing plates were used to support a DSTA specimen at the ends. These bearing plates were designed with the objective of versatility to accommodate different specimen sizes with diameters within the range of 100 mm to 200 mm at an angle of 29°±5° with respect to the horizontal direction. A threaded bar was used for minute angle changes of the adjustable bearing plate, which allowed for the nearly perfect seating of the arch at the supports. A bearing plate with a DSTA in place is shown in Figure 4b. Floor brackets (Figures 4a and 4c)
were used to stop any out-of-plane movements while allowing in-plane movements. Loading was applied to the arch using a clamp system attached to the actuator head (Figure 4d).

![Overall view of experimental setup](image1)
![Bearing plate with an arch in place](image2)
![Floor bracket placed at mid-span](image3)
![Clamp attached to the specimen and the actuator head](image4)

**Figure 4 Experimental loading setup**

**Instrumentation and Loading**

Strain gauges were attached to both the steel tube and the FRP tube to measure axial and hoop strains. In total, five sections were selected for measurement (Figure 5a), including the mid-span section (L1), sections of 200 mm (L2), 300 mm (L3) and 800 mm (L4) offset from the mid-span section, and the section at 100 mm offset from the support (L5). The strain gauge arrangement on each section is shown in Figure 5b. Considering the symmetry conditions, the strain gauges were placed only on one half of the section. Ten strain gauges, including five in the axial direction (denoted by “A” in Figure 5b) and 5 in hoop direction (denoted by “H” in Figure 5b), were installed on the FRP tube at each section to capture the axial strain and hoop strain distributions at each section and also along the arch axis. In addition, the strains of the steel tube were measured by four strain gauges at each section, including two in the hoop direction and two in the axial direction.

To obtain the deformation curve of each arch during the loading process, seven symmetrically-arranged linear position transducers (Figure 5c) were employed to measure the displacements of the arch at different locations. Actuator readings were taken for both the load and the crown displacement. Loading was applied at a rate of around 2 mm/min.

![Selected sections for strain measurement](image5)
![Strain gauge layout](image6)
![Linear position transducer arrangement](image7)

**Figure 5 Layout of strain gauges and linear position transducers**

**TEST RESULTS AND DISCUSSIONS**

**Load-Deflection Responses**

The curves of load vs mid-span deflection of all three specimens are shown in Figure 6. The initial stiffnesses of all the specimens are similar, but as the load increases, the stiffnesses of specimens 4P904E and 6P904C tend to decrease faster than that of specimen 4P904C. The faster decreases in stiffness of specimen 4P904E are believed to be due to the eccentric placement of the steel tube, resulting in a reduced section rigidity in the hogging moment regions. The faster decreases in stiffness of specimen 6P904C are believed to be due to manufacturing defects during the wet-layup process.

Of the three specimens, specimen 4P904C achieved the highest load-carrying capacity, with an ultimate load of 259 kN (referring to the sum of the two point loads in the present paper). Specimens 6P904C and 4P904E
attained ultimate loads of 230 kN and 170 kN respectively. All three specimens were able to undergo large deformations without substantial reductions in the load. The unloading stiffnesses of all three specimens were found to be similar to the respective initial stiffnesses. During the testing of specimen 6P904C, a sudden load drop accompanied by a loud noise occurred, after which the specimen could be reloaded to its ultimate load. Specimen 4P904E also experienced a small load drop close to its ultimate load.

**Failure Modes**

In all three specimens, tensile cracks of the FRP tube in the fibre direction were observed (Figure 7). These tensile cracks initiated during an early stage of loading (at a load of approximately 40 kN) near the mid-span (Figure 7a); more cracks of the same nature appeared in other regions of the arch (within the region of approximately 1100mm to 1400mm from the mid-span on each side) as the load continued to increase (Figures 7b and 7c). Naturally, these tensile cracks on the GFRP tube were always in the tensile region of the cross-section and their horizontal locations are related to the distribution of bending moment along the arch.

After the test, the GFRP tube was cut open to inspect the concrete annulus. Near the tensile cracks, concrete cracking was observed. However, these concrete cracks were localized in the cracked regions of the GFRP tube, and no cracks were observed in the compression region of the cross-section. Upon unloading, significant plastic deformation of the arch was observed for all three specimens (Figure 7c).

**Axial Strains**

Due to space limitations, only the axial strains of selected locations are discussed in this paper. The axial strains of the FRP tube at sections L3 and L4 of specimen 4P904C are selected to represent the sagging moment and the hogging moment regions respectively and are given in Figures 8a and 8b. The strain readings at section L4 of the steel tube are given in Figure 8c.

Axial strains of the FRP tube were found to vary over the cross-section of the arch at both sections L3 and L4, obviously as a result of bending effects. At section L3, the top part of the section (i.e. points C1 & C2 in Figure 5b) are seen to have high compressive strains while points C3-C5 had relatively low strain values. At section L4,
the top part of the section (points C1 & C2) showed low tensile strains, while the bottom part (points C4-C5) showed relatively high compressive strains. Similar trends were observed in specimens 6P904C and 4P904E.

Similar to the FRP tube, the steel tube had axial strains that vary significantly over the cross-section due to bending effects. A clear change in the strain increase rate is seen when the load is close to 220kN (Figure 8), indicating the yielding of the steel tube. However, the strain readings on the steel tube at this load were higher than the yield strain based on the manufacturer’s data. Steel coupon tests are yet to be carried out in order to obtain the actual material properties for the steel tubes.

### Hoop Strains

Hoop strains at sections L3, L4 and L5, shown in Figure 9, are seen to vary over the cross-section as well. The hoop tensile strain values were generally higher where the axial compressive strains were higher; the hoop tensile strains reduce in value or even change to be compressive strains down the section height (i.e., point C5 at section L3 and point C1 at section L3). At section L5, while the hoop strains were non-uniform over the cross-section, all hoop strain values were positive (i.e., GFRP tube was in tension around the entire circumference) at higher loads.

### Discussions

The test results showed that the hybrid double-skin tubular section can function effectively in arches. The test DSTAs showed a highly ductile response, which cannot be expected of similar steel-concrete arches without the external GFRP tube. Increasing the FRP volume from 4100 g/m² (4P904C) to 6150 g/m² (6P904C) did not increase the load-carrying capacity of the arch. In fact, specimen 6P904C had a load-carrying capacity that is 11.2% lower than that of specimen 4P904C. This may have been caused by the difference in the manufacturing technique, with a better quality control for the GFRP tube in specimen 4P904C. The eccentricity of the steel tube resulted in a reduction in the load-carrying capacity. Eccentric placement of the steel tube increases the sagging moment capacity but reduces the hogging moment capacity of the cross-section, leading to an overall effect of reduction in the load-carrying capacity of the DSTA.

### CONCLUSIONS

This paper has presented the results of three tests on hybrid double-skin tubular arches (DSTAs) under two concentrated loads near the mid-span (four-point bending tests). A DSTA is composed of a steel inner tube, an FRP outer tube, and a concrete infill between the two tubes. The main parameters examined in this study include...
the number of layers forming the FRP tube and the section configuration. Based on the test results and discussions, the following conclusions may be drawn:
(1) DSTAs have much higher load-carrying capacities than similar steel-concrete arches without an FRP tube;
(2) DSTAs possess a very ductile response;
(3) The DSTA specimen with an eccentrically-placed steel tube had a lower load-carrying capacity than its counterpart with a concentrically-placed steel tube;
(4) Increasing the GFRP tube thickness from 4 layers to 6 layers did not increase the load-carrying capacity; instead a reduction was observed. This could be attributed to defects during the manufacturing of the DSTA with a 6-layer GFRP tube.
(5) Further research is necessary to better understand and model the behaviour of DSTAs.

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LOCAL BUCKLING OF HYBRID FRP-TIMBER THIN-WALLED COLUMNS: AN EXPERIMENTAL INVESTIGATION

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ABSTRACT

Recent research at the University of Queensland (UQ) has led to the development of a new type of structures called “hybrid fibre reinforced polymer (FRP)-timber (HFT) structures”. In HFT structures, FRP is combined with timber veneers to create high-performance, lightweight, easy-to-construct structural members. These HFT members utilize the orthotropic properties of both timber and FRP in a complementary combination to produce much improved composite properties and to maximise the load-carrying capacity of members for a given amount of material. While preliminary experimental work has demonstrated the potential of HFT sections as high-performance sustainable structural components, much more work is needed to better understand the behaviour of HFT structures. This paper presents the results of an experimental study into the local buckling behaviour of HFT thin-walled Cee section short columns. The experimental programme consisted of fifteen HFT column specimens, including all-timber columns and three different types of HFT columns. The test results are presented and discussed. HFT Cee section short columns carried significantly higher axial loads than the corresponding all-timber columns. The ultimate load-to-weight ratio of the HFT sections is shown to be comparable to or significantly higher than that of cold-formed thin-walled steel Cee sections.

KEYWORDS

Timber, FRP, Hybrid FRP-timber columns, local buckling, thin-walled structures, lightweight structures.

INTRODUCTION

Lightweight steel and aluminium thin-walled structures have become popular in the construction industry due to their advantages such as ease for assembly, transportation and installation. Steel or aluminium thin-walled lightweight structural sections, manufactured using the cold-forming technique, often exist as open profiles such as Cee or Zee sections. The load-carrying capacity of these members is typically governed by buckling failure in a local, distortional or global buckling mode. While these structural members possess many advantages, steel and aluminium are greenhouse gas-intensive materials. Therefore, steel and aluminium thin-walled structural members have been considered as non-sustainable (Yan et al. 2010).

At the University of Queensland (UQ), much recent research has been devoted to the development of high value end uses of low-quality timber products, which has resulted in the development of a new form of composite structures. These new structures, termed hybrid fibre reinforced polymer (FRP)-timber (HFT) thin-walled structures, are formed by combining FRP with thin veneers to create high-performance, lightweight, easy to construct structural members. These new products have many potential applications such as roof purlins, floor panels, stud walls, facades, load-bearing walls, bridge decks, etc. These new products utilize the orthotropic properties of timber and FRP in a complementary combination: the fibre directions are appropriately oriented to produce near-optimal composite properties and to achieve cross-sectional shapes that maximize the load-bearing capacity for a given amount of material. The grain orientations of the veneer in the section shown in Figure 1 are approximately parallel to the longitudinal axis while the fibres of the FRP layers are oriented perpendicular to the longitudinal axis. These HFT sections take advantage of the veneer’s natural tendency to curl towards one side, making the formation of open sections easily achievable around a mandrel, with the veneer grains oriented in the direction of the member longitudinal axis. However, due to this orientation of the veneer grains (parallel to the longitudinal axis), HFT sections would have weak properties in the transverse direction if no FRP layers were provided. Therefore, in these HFT sections, FRP layers with the fibres oriented predominantly in the
transverse direction are provided to enhance the transverse properties. The presence of FRP layers can significantly enhance the buckling strength of HFT thin-walled members.

Figure 1 HFT Cee-section (arrow heads indicate the FRP fibre and timber grain directions)

HFT sections take advantage of two existing novel technologies, namely the use of FRP composites to reinforce low-quality timber products (Buell and Saadatmanesh 2005; Fernando et al. 2016) and veneer-based innovative timber sections made from hardwood thinnings (Gilbert et al. 2014). A preliminary experimental investigation showed that HFT wall panels can significantly outperform all-timber wall panels in terms of axial load-carrying capacity (Fernando et al. 2015). The recent study of Gilbert et al. (2014) showed that all-timber thin-walled structures have a compressive capacity-to-weight ratio of up to 30% greater than cold-formed steel structures, while having a much higher bending stiffness (up to 3.3 times higher). These observations clearly suggest that HFT sections can be expected to perform significantly better than cold-formed steel columns in terms of load-carrying capacity-to-weight ratio. Therefore, HFT sections are believed to have great potential in structural applications.

This paper presents an experimental study into the local buckling failure of HFT Cee sections under axial compression. The test program consisted of fifteen short column specimens, including three all-timber columns and twelve HFT columns. The column specimens were constructed of softwood (hoop pine) veneers and FRP layers; different FRP materials were explored in the study, including glass FRP (GFRP), natural fibre FRP (NFRP), and natural fibre mat FRP (NMFRP).

EXPERIMENTAL PROGRAMME

General

In the present study, fifteen HFT Cee section column specimens were manufactured and tested under axial compression. These specimens included three all-timber columns, four each of hybrid GFRP-timber (HGFT), hybrid NFRP-timber (HNFT), and hybrid NMFRP-timber (HNMFT) columns. The NFRP used linen fabrics produced from fibres extracted from the flax plant while the NMFRP used fibres extracted from the Kenaf plant. For ease of reference, the name of each column specimen starts with a letter of “A” - “D” representing the all-timber, HGFT, HNFT, and HNMFT specimens respectively, followed by a Roman number to differentiate the four nominally identical specimens of each type. For an example, specimen A-I means the first specimen of all-timber Cee section columns.

Figure 2 HFT Cee-section column

The nominal dimensions of the cross-section for the test columns are shown in Figure 2a. The internal depth, internal width and internal radius of the cross-section were 150 mm, 73 mm and 15 mm respectively. Each
specimen consisted of three 0.7 mm nominal thickness hoop pine timber veneer layers as the core and a number of FRP layers applied on the inner and outer surfaces of the timber core: (a) two GFRP layers, with one each on the inner and outer surfaces of the timber core in the HGFT specimens, (b) six NFRP layers, with three each on the inner and outer surfaces of the timber core in the HNFT specimens, and (c) two NMFRP layers, with one each on the inner and outer surfaces of the timber core in the HNMFT specimens. The numbers of NFRP and NMFRP layers were selected to match the weight of GFRP layers used in the HGFT specimens. Preliminary finite element (FE) analysis was carried out to determine the elastic buckling modes of the columns. Based on the FE results, a height of 280 mm was selected for the column specimens to ensure that local buckling would govern the failure. The measured thicknesses of the column specimens are given in Table 1. As the different fibres had significantly different absorption rates for the adhesive, some variations in the thickness were observed. These thickness variations were enlarged by the different volumes of adhesive absorbed by the filter cloth used during the manufacturing process. In particular, HNMFT specimens showed significant variations in the thickness.

![Figure 3 Curing of column specimen under vacuum pressure](image)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Type of FRP</th>
<th>Average thickness (mm)</th>
<th>Weight (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-I</td>
<td>N/A</td>
<td>2.50</td>
<td>134</td>
</tr>
<tr>
<td>A-II</td>
<td>N/A</td>
<td>2.69</td>
<td>126</td>
</tr>
<tr>
<td>A-III</td>
<td>N/A</td>
<td>2.62</td>
<td>132</td>
</tr>
<tr>
<td>B-I</td>
<td>GFRP</td>
<td>6.14</td>
<td>329</td>
</tr>
<tr>
<td>B-II</td>
<td>GFRP</td>
<td>4.32</td>
<td>284</td>
</tr>
<tr>
<td>B-III</td>
<td>GFRP</td>
<td>4.09</td>
<td>267</td>
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<tr>
<td>B-IV</td>
<td>GFRP</td>
<td>3.51</td>
<td>268</td>
</tr>
<tr>
<td>C-I</td>
<td>NFRP</td>
<td>7.44</td>
<td>470</td>
</tr>
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<td>NFRP</td>
<td>6.85</td>
<td>458</td>
</tr>
<tr>
<td>C-III</td>
<td>NFRP</td>
<td>6.97</td>
<td>463</td>
</tr>
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<td>C-IV</td>
<td>NFRP</td>
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</tr>
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<td>NMFRP</td>
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<td>NMFRP</td>
<td>9.56</td>
<td>546</td>
</tr>
<tr>
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<td>NMFRP</td>
<td>6.35</td>
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<tr>
<td>D-IV</td>
<td>NMFRP</td>
<td>9.00</td>
<td>506</td>
</tr>
</tbody>
</table>

**Specimen Preparation**

In all the column specimens, the grain orientation of the veneers was aligned with the column longitudinal axis. In the HGFT and HNFT specimens, the fibre orientation of FRP was perpendicular to the longitudinal axis, while the NMFRP had fibres oriented at random directions. In all the HFT specimens, the FRP layers were expected to enhance the transverse properties of the timber. The specimen preparation process started with the bonding of veneers and FRP layers together around a mandrel, followed by the application of pressure using a vacuum bag to maintain the shape until the adhesive had cured properly (Figure 3). During the preparation process, filter clothes were used between the vacuum bag and the Cee section external surface to absorb any excess adhesive. Based on the recommendations of Miao et al. (2015), a commercially available polyurethane adhesive, PURBOND, was used to bond the layers together in all specimens. The curing time recommended by the manufacturer for the PURBOND adhesive was 4 hours. After 4 hours, the vacuum bag was removed and the specimen was removed from the mandrel. Each specimen was initially 400 mm in height, but was cut to a height of 280 mm upon full curing. The offcuts were used to obtain coupons for material testing.
Material Properties

The mechanical properties of the HFT sections were determined by tensile coupon tests. Ten 100 mm *10 mm coupon specimens, five for each direction (parallel and perpendicular to the column longitudinal axis respectively) were cut from the web of the offcuts. Two aluminium tabs of 25 mm in length were attached to each end of a coupon specimen to avoid failure within the gripping regions. The dimensions of the coupons were selected as per the recommendations given in ASTM-D3039/D3039M (2014). The coupons were then loaded in axial tension using a 100 kN capacity INSTRON testing machine. A video extensometer was used to obtain the longitudinal and transverse strains during the loading while the load was obtained from the testing machine. The average mechanical properties from coupon tests for the column specimens, in terms of tensile stiffness (Et) and tensile strength per unit width (σu), are given in Table 2.

Table 2 Mechanical properties of column specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Type of FRP</th>
<th>Et perpendicular to axis (N/mm)</th>
<th>Et parallel to axis (N/mm)</th>
<th>σu perpendicular to axis (N/mm)</th>
<th>σu parallel to axis (N/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-I</td>
<td>N/A</td>
<td>1472.5</td>
<td>21477.5</td>
<td>9.6</td>
<td>138.1</td>
</tr>
<tr>
<td>A-II</td>
<td>N/A</td>
<td>1121.7</td>
<td>17635.6</td>
<td>9.3</td>
<td>141.2</td>
</tr>
<tr>
<td>A-III</td>
<td>N/A</td>
<td>958.9</td>
<td>15735.7</td>
<td>7.3</td>
<td>114.4</td>
</tr>
<tr>
<td>B-I</td>
<td>GFRP</td>
<td>34998.0</td>
<td>28594.0</td>
<td>341.6</td>
<td>146.6</td>
</tr>
<tr>
<td>B-II</td>
<td>GFRP</td>
<td>27751.7</td>
<td>31557.6</td>
<td>414.7</td>
<td>231.9</td>
</tr>
<tr>
<td>B-III</td>
<td>GFRP</td>
<td>26535.9</td>
<td>27513.4</td>
<td>318.9</td>
<td>143.2*</td>
</tr>
<tr>
<td>B-IV</td>
<td>GFRP</td>
<td>26693.6</td>
<td>26616.3</td>
<td>386.1</td>
<td>186.0</td>
</tr>
<tr>
<td>C-I</td>
<td>NFRP</td>
<td>24857.0</td>
<td>21769.4</td>
<td>297.6*</td>
<td>416.6*</td>
</tr>
<tr>
<td>C-II</td>
<td>NFRP</td>
<td>20289.7</td>
<td>21666.6</td>
<td>363.1</td>
<td>246.6</td>
</tr>
<tr>
<td>C-III</td>
<td>NFRP</td>
<td>24248.6</td>
<td>20289.7</td>
<td>313.7</td>
<td>179.7</td>
</tr>
<tr>
<td>C-IV</td>
<td>NFRP</td>
<td>25660.3</td>
<td>25642.7</td>
<td>442.5</td>
<td>291.1</td>
</tr>
<tr>
<td>D-I</td>
<td>NMFRP</td>
<td>6219.7</td>
<td>28494.2</td>
<td>74.5</td>
<td>230.0</td>
</tr>
<tr>
<td>D-II</td>
<td>NMFRP</td>
<td>4521.9</td>
<td>30716.3</td>
<td>70.6</td>
<td>248.6*</td>
</tr>
<tr>
<td>D-III</td>
<td>NMFRP</td>
<td>6477.0</td>
<td>27997.2</td>
<td>98.9</td>
<td>243.2</td>
</tr>
<tr>
<td>D-IV</td>
<td>NMFRP</td>
<td>5283.0</td>
<td>29520.0</td>
<td>117.0*</td>
<td>306.0*</td>
</tr>
</tbody>
</table>

* Failure occurred in the grips; E = elastic modulus; σu = tensile strength

Test Set-up and Instrumentation

All the column tests were carried out using a 250 kN capacity INSTRON testing machine. The test set-up is shown in Figure 4. The specimen was seated on a fixed steel base plate, and the load was applied at the top through a steel plate with a hemispherical block to allow rotational movements between the machine loading head and the top steel plate. When placing a specimen in the testing machine, the centre of the hemispherical
block was aligned with the centroid of the specimen cross-section to ensure concentric axial loading. The axial load was applied with displacement control at a rate of 0.2 mm/min.

Three linear variable displacement transducers (LVDTs) were used to measure the horizontal column displacements at a specific height (i.e., mid-height or quarter height) where the buckling mode was expected to have its maximum displacement according to finite element analysis. A non-contact 3D optical method (DIC system) was employed to capture the deformed shape of the inter surface of one of the flanges.

RESULTS AND DISCUSSIONS

All column specimens failed due to local buckling. The typical buckling mode observed for each specimen type is shown in Figure 5. Compared to type A specimens, a significant reduction in the half-wave length was observed in specimen types B and C (Figure 5). In the HFT specimens, the lateral movements of the flanges were often concentrated in either the lower or the upper portion of the column (Figures 5b and 5c) instead of closely following a sinusoidal pattern. This is attributed to the inherent non-uniformity of material and/or geometric properties down the column height. Upon removal of the load, all specimens bounced back to being close to their respective original shapes, showing little to no damage.

![Figure 5 Typical buckling modes of different specimen types before removal of axial load](image)

The load-axial displacement curves obtained from the testing machine for specimens “A” - “D” are given in Figure 6. In terms of the initial stiffness, the HFT specimens showed better consistency than the all-timber specimens. The HFT specimens also had a much higher initial stiffness than that of the all-timber specimens. The differences in the average initial stiffness amongst the different types of HFT specimens are small.

![Figure 6 Load-displacement curves of all column specimens](image)

The ultimate loads of the column specimens are given in Table 3. The addition of FRP layers significantly increased the local buckling capacity of the column. The highest load increase was achieved by type D specimens, with the average ultimate load increase being 276% over type A specimens. The average ultimate loads of types B and C specimens are 158% and 267% higher than that of type A specimens. Types A and B specimens had average measured weights of 0.131 kg and 0.287 kg, respectively. The average ultimate load per
unit weight for type B specimens is 17% higher than that of type A specimens. Although the weights, thus the volumes of adhesive of the four type B specimens were different, the ultimate loads showed limited differences. Similar observations can be made about types C and D specimens.

Table 3 Results of column compression tests

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Type of FRP</th>
<th>Ultimate load (kN)</th>
<th>Average ultimate load (kN)</th>
<th>Average specimen weight (kg)</th>
<th>Average ultimate load per unit weight (kN/kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-I</td>
<td>N/A</td>
<td>6.58</td>
<td>6.39</td>
<td>0.131</td>
<td>48.78</td>
</tr>
<tr>
<td>A-II</td>
<td>N/A</td>
<td>6.03</td>
<td></td>
<td>0.287</td>
<td>57.35</td>
</tr>
<tr>
<td>B-I</td>
<td>GFRP</td>
<td>16.41</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-II</td>
<td>GFRP</td>
<td>17.01</td>
<td>16.46</td>
<td>0.287</td>
<td>57.35</td>
</tr>
<tr>
<td>B-III</td>
<td>GFRP</td>
<td>16.25</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-IV</td>
<td>GFRP</td>
<td>16.16</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C-I</td>
<td>NFRP</td>
<td>25.59</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C-II</td>
<td>NFRP</td>
<td>23.67</td>
<td>23.42</td>
<td>0.453</td>
<td>51.70</td>
</tr>
<tr>
<td>C-III</td>
<td>NFRP</td>
<td>23.66</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>C-IV</td>
<td>NFRP</td>
<td>20.76</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D-I</td>
<td>NMFRP</td>
<td>23.12</td>
<td></td>
<td></td>
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<tr>
<td>D-II</td>
<td>NMFRP</td>
<td>24.46</td>
<td>24.02</td>
<td>0.483</td>
<td>49.73</td>
</tr>
<tr>
<td>D-III</td>
<td>NMFRP</td>
<td>21.65</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D-IV</td>
<td>NMFRP</td>
<td>26.85</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

While the HFT Cee section columns achieved significantly higher ultimate loads than the all-timber Cee section columns, the average ultimate load per unit weight of the HFT Cee section columns is only marginally higher than that of the all-timber Cee section columns. Since only three 0.7mm thick timber veneers were used in the present HFT Cee section columns, the weight of the FRP layers was higher than that of the timber veneers in these HFT sections. However, in HFT sections with a significantly higher timber mass ratio, the ultimate load per unit weight can be expected to be increase accordingly. HFT wall panels tested by Fernando et al. (2015) showed a 91% increase in the average ultimate load due to the addition of FRP while the average weight increase was less than 28%.

Table 4 Comparison between HFT sections and cold-formed steel sections

<table>
<thead>
<tr>
<th>Type</th>
<th>Average ultimate load (kN)</th>
<th>Average column weight per 280mm in length (kg)</th>
<th>Average ultimate load per unit weight (kN/kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>6.39</td>
<td>0.131</td>
<td>48.78</td>
</tr>
<tr>
<td>B</td>
<td>16.46</td>
<td>0.287</td>
<td>57.35</td>
</tr>
<tr>
<td>C</td>
<td>23.42</td>
<td>0.453</td>
<td>51.70</td>
</tr>
<tr>
<td>D</td>
<td>24.02</td>
<td>0.483</td>
<td>49.73</td>
</tr>
<tr>
<td>SC120x60</td>
<td>36.49</td>
<td>0.806</td>
<td>45.25</td>
</tr>
<tr>
<td>SC180x60</td>
<td>37.91</td>
<td>0.994</td>
<td>38.14</td>
</tr>
</tbody>
</table>

**COMPARISON WITH COLD-FORMED STEEL**

Mulligan (1983) tested cold-formed steel Cee section columns under axial compression. Specimens SC120x60 (156mm in depth, 76mm in width, 1.2mm in thickness and 474 mm in height) and SC180x60 (225 mm in depth, 76 mm in width, 1.21 mm in thickness and 457 mm in height) were found to have axial load capacities of 36.49 kN and 37.91kN respectively (Table 1). These columns failed by local buckling, so it is reasonable to assume that the ultimate load depends little on colour height. Specimen SC120x60 weighed 0.806 kg per 280 mm in length while specimen SC180x60 weighed 0.994 kg per 280 mm in length, both being much heavier than the present HFT sections. The axial load capacities per unit weight of the HFT section columns (with a height of 280 mm) exceed...
those of the SC120x60 and SC180x60 section columns, confirming that the HFT sections can perform significantly better than similar standard cold-formed steel sections in terms of ultimate load-weight ratio.

CONCLUSIONS

This paper has presented an experimental investigation into the buckling behaviour of hybrid FRP-timber (HFT) thin-walled Cee section short columns. The 280 mm long HFT columns were formed by adhesive bonding of timber veneers and FRP layers. The performance of HFT columns made with three different types of FRPs [namely, glass FRP (GFRP), natural fibre FRP (NFRP), and natural fibre mat FRP (NMF FRP)] were investigated. Tensile coupon tests were carried out to determine the material properties for each column specimen. The column compression test results showed that: (i) HFT specimens performed much better than all-timber specimens in terms of both load-carrying capacity and axial stiffness; (ii) NMHFT specimens achieved the highest axial load capacity; (iii) GHFT specimens had the highest axial load capacity per unit weight; and (iv) the load-carrying capacity per unit weight of HFT sections can be significantly higher than that of lightweight thin-walled cold-formed steel sections. The last conclusion means that HFT sections can be used as an attractive sustainable alternative to cold-form steel sections in practice. While the load-carrying capacity per unit weight of the present HFT sections are only marginally higher than that of the present all-timber sections, it can be expected to become significantly higher in sections with a higher timber mass ratio. Further investigations are required to study the different buckling failure modes of HFT columns and to develop numerical/analytical models to predict the behaviour of HFT columns.

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REFERENCE:


EXPERIMENTAL RESEARCH ON GFRP-CONCRETE-STEEL COMPOSITE BEAMS

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ABSTRACT

A novel GFRP-concrete-steel composite beam was proposed to improve durability, which is consisted of GFRP-concrete composite deck and steel girder and then tested under the static loads. In our study, two GFRP-concrete-steel composite beams with different GFRP-concrete interface were fabricated and a traditional steel-concrete composite beam was used as a control sample. The deformation, bending performance and failure modes of the model beams were discussed and compared. The experimental results show that all specimens are failed with concrete crush. Both GFRP-concrete-steel composite beams had lower stiffness than that of traditional steel-concrete composite beam. However, the slip effects of GFRP-concrete-steel composite beams were similar to the effect of traditional steel-concrete composite beam. Moreover, different GFRP-concrete interface have no effect on the experimental results of GFRP-concrete-steel composites. Also the consistence of our theoretical results with experimental results demonstrated that the ultimate capacity of GFRP-concrete-steel composite beams could be predicted by the bending bearing capacity calculation method based on considering elastic-plastic performance.

KEYWORDS

FRP, composite beams, GFRP-concrete deck, slip, ultimate capacity, deformation.

INTRODUCTION

Highway bridges, especially bridge decks, are exposed to environment erosion and overweight vehicles, which increase the costs at the later stage of maintenance. In order to improve the durability of bridge structure, increase fatigue and corrosion resistance, prolong the service life, fiber reinforced polymer (FRP) has been applied in different components of bridges, such as: CFRP Cable, GFRP beam, GFRP bridge deck. Recently, GFRP-concrete composite bridge deck has been concerned by many researchers as a method for improving the durability of concrete bridge deck, which has the characteristics of corrosion resistance, easy construction, low maintenance cost (Nelson et al. 2014). At present, the research efforts on the interface between GFRP and concrete, the cross section types of GFRP and the static performance of composite deck are reported. The types of interface include wet bond interface, coarse sand coated, combined shear connector system, etc. (Honickman et al. 2009, Cho et al. 2010). According to different forms of composite slabs, the cross section types of GFRP may include T-up stiffeners on plate, dual cavity system, tubular sections on plate, etc. (Dieter et al. 2002, Nelson et al. 2014). The static tests cover one-way simply supported slabs, two-way slabs, continuous slabs (Alagusundaramoorthy et al. 2006, Huang et al. 2015).

The composite girder bridge with GFRP-concrete deck has better resistance to the overweight vehicles and environment erosion. Although experimental and analytical work on the mechanical characteristics in the transverse bridge direction have been widely reported, the research on the composite action of GFRP-concrete deck and steel girder is rare. This paper presents the experimental results of GFRP-concrete-steel composite beams which consist of GFRP-concrete deck with GFRP groove plate and steel beam. The samples include two GFRP-concrete-steel composite beams with different surface treatment (with or without wet bonding technique) and a steel-concrete composite beam as a contrast specimen. The bending mechanical characteristic of composite beams was tested. The tests emphasize on strain distribution across section, load and displacement relationship, slip value from support to mid-span. In the end, the bending failure load was calculated by elastic-plastic calculation method. The prediction agreed well with test data.
EXPERIMENTAL PROGRAM

Test Specimens

Three composite beams, referred to as S-1 to S-3, were fabricated and tested. Table 1 provides a summary of the test beams, including dimensions, calculated spans, reinforcement content, pitch of shear stud, GFRP-concrete interface. The GFRP panel is consisted of T-up stiffeners and bottom plate (Figure 1(c)). The three composite beams were tested to evaluate the influence of GFRP. S-1 (with wet bonding technique) and S-3 (without wet bonding technique) are both GFRP-concrete-steel composite beams. S-2 is a steel-concrete composite beam with same size and material. Figure 1 shows the cross sections of composite beams and GFRP.

![Figure 1 test specimens (mm)](image)

<table>
<thead>
<tr>
<th>specimen</th>
<th>S-1</th>
<th>S-2</th>
<th>S-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab size (mm)</td>
<td>88×500×3000</td>
<td>86×500×3000</td>
<td>93×500×3000</td>
</tr>
<tr>
<td>calculated span (mm)</td>
<td>3000</td>
<td>3000</td>
<td>3000</td>
</tr>
<tr>
<td>Steel top flange(mm)</td>
<td>160×12</td>
<td>160×12</td>
<td>160×12</td>
</tr>
<tr>
<td>Steel web(mm)</td>
<td>180×12</td>
<td>180×12</td>
<td>180×12</td>
</tr>
<tr>
<td>Steel bottom flange(mm)</td>
<td>180×8</td>
<td>180×8</td>
<td>180×8</td>
</tr>
<tr>
<td>Longitudinal bars</td>
<td>8@6</td>
<td>8@6</td>
<td>8@6</td>
</tr>
<tr>
<td>Pitch of shear stud(mm)</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Diameter of shear stud(mm)</td>
<td>13</td>
<td>13</td>
<td>13</td>
</tr>
<tr>
<td>Type of applied loading</td>
<td>Two-point</td>
<td>Two-point</td>
<td>Two-point</td>
</tr>
<tr>
<td>GFRP-concrete interface</td>
<td>Wet bonded</td>
<td>—</td>
<td>Unbonded</td>
</tr>
</tbody>
</table>

NOTE: The concrete casting thickness was thicker than the designed thickness a little. The slab sizes in table 1 are measured values.

Materials

The concrete was fabricated, designed to have a compressive strength of 23.4 MPa at 28 days. The measured strengths based on test cubes from S-1 to S-3 were 24.2 MPa, 26.1 MPa and 27.1 MPa. The steel coupons cut from the bottom flange and web with a thickness of 12mm and 8mm were measured. The yield strengths of 428~502 MPa, the ultimate strengths of 554~568 MPa were obtained. 6 mm diameter bars were used as longitudinal reinforcement with yield strength 440 MPa. The compressive strength and modulus of GFRP in the transverse direction were 103.5 MPa and 10.8×10^3 MPa. The tensile strength and modulus of GFRP were 80.9 MPa and 11.48×10^3 MPa.

Fabrication of Test Specimens, Loading and Instrumentation

The fabrication process were as follows: 1) GFRP formwork panels were fabricated in the factory. 2) GFRP panels were placed directly on the steel girders. During setting, the panels were bonded together at the panel-to-panel splice location using epoxy. 3) According to the design, the steel bars were installed (Figure 3). 4) For S-1, the GFRP bottom plate was coated with a thin layer of epoxy resin no more than 50 minutes before casting concrete while the epoxy was still wet. For S-3, concrete was poured directly onto the GFRP formwork.

Four-point bending tests were performed using hydraulic jack with capacities of 1000 kN (Figure 4). The clear span was 3 m and the loading points were 0.4 m. The deflection at the mid-span and support were measured by displacement gauges. Moreover, five displacement gauges were used to measure the slip between the GFRP-concrete deck and steel beam. The stress of concrete, steel bars, steel beams, GFRP were measured by the strain gauges as shown in Figure 5. The deflection and strains were recorded with Donghua 3816N data acquisition system.
EXPERIMENTAL RESULTS AND DISCUSSION

Failure Modes

The decks of the composite beams were all compressive in the process of static test. The failure modes were all concrete crushed (Figure 6). When the load reached 415 kN, the concrete on the top of T-up stiffeners began to spall for S-1. As the load reached 440 kN, the concrete was crushed. For S-3, the failure mode was similar. When the load reached 440 kN, the concrete on the top of T-up stiffeners began to spall. When the load reached 465 kN, the concrete was crushed. There was a little difference for S-2 without GFRP. The concrete began to spall at 430 kN, the concrete was crushed at 435 kN. Because of the confining reactions provided by GFRP the buckling deformation of the steel bars were not large, the GFRP-concrete-steel composite beam achieved higher load after the top concrete began to spall. However, the reinforcing bars of the steel-concrete beam losing the efficiency of the confinement had a large buckling deformation after the top concrete spalled.

Load-Deflection Relationship

The load-deflection relationship is shown in Figure 7. When the load was small, the girder response was linear. Beyond 60% to 70% of the ultimate load, the curve became progressively nonlinear. The large deformation of the composite beams at ultimate load showed that the three specimens had good ductility. The deformation of specimen S-2 is smaller than S-1 and S-3 in Figure 7, because the modulus of GFRP is lower than the modulus of concrete.

Effect of Slip

Figure 8 shows the interface slip distributions between GFRP and steel. The curves of the three specimens were similar. The value of slip at mid-span was minimum. After a rapid increase the slip curve became smooth. The maximum value of the slip occurred near the support, not the bearing point. Because the loads were not absolutely symmetric and the measuring error couldn’t be eliminated, the value of the slip at mid-span was not zero.
Strain Distribution

Figure 9 shows the strain distribution of the three specimens at mid-span. The strain gauges position is shown in Figure 5. At the beginning of the test, the value of slip at the interface of GFRP and steel can be ignored. When the load was closed to the failure load, the value of slip increased rapidly. As the top concrete began to spall near the ultimate load, the concrete stain gauges of specimens S-1 and S-3 were failure. The top strain was obtained by extending the connecting line of GFRP and reinforcing bar strains. When the load reached 97% of ultimate load, the strains of GFRP and reinforcing bars were proportional to the distance from the neutral axis and the curvature of GFRP-concrete deck and steel girder were close. The top strain of specimens S-1 and S-3 were roughly 4950 με and -5435 με, respectively. These were higher than the ultimate strain of unconfined concrete. It is because the GFRP has a confining effect on concrete. This effect may be similar to concrete confined by rectangular hoops. The longitudinal reinforcing bars confined by transverse steel and the larger bar diameter of transverse bars which provides greater flexural stiffness increase the confining effect (R.Park and T.Paulay 1975). The T-up stiffeners have confined the longitudinal reinforcing bars and provide greater flexural stiffness. Therefore, the ultimate strain on the top of concrete is greater.

Analysis of Ultimate Bending Capacity

Based on the results of the test, it is assumed that:
1) Plane sections before bending remain plane after bending, the slip effect is ignored (Nie and Shen 1997); 2) The ultimate strain on the top of concrete is assumed as 0.0035; 3) The neutral axis is in the steel beam; 4) Concrete and steel are elastic-plastic material perfectly; 5) The stress of concrete on the section at mid-span all can reach ultimate stress.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>The distance of neutral axis from top surface (mm)</th>
<th>Bending capacity (kN•m)</th>
<th>Calculation / Test</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Calculation</td>
<td>Test</td>
<td>Calculation</td>
</tr>
<tr>
<td>S-1</td>
<td>117</td>
<td>101.7</td>
<td>273.6</td>
</tr>
<tr>
<td>S-2</td>
<td>106.9</td>
<td>87.3</td>
<td>281</td>
</tr>
<tr>
<td>S-3</td>
<td>114.4</td>
<td>106.8</td>
<td>286.3</td>
</tr>
</tbody>
</table>

Following the plane cross-section assumption and internal force equilibrium, the elastic-plastic bending load is calculated (Huang 2004). Table 6 shows the results with elastic-plastic calculation method. The theoretical results of specimen S-2 agree well with the measured values. The results of specimens S-1 and S-3 are higher than the measured values, because the top strain is higher than 0.0035. In addition, table 6 shows the theoretical neutral axis position and measuring neutral axis position. It is in the steel web for specimens S-1 and S-3. For specimen S-2, it is in the top flange. The measuring neutral axis position is higher than the theoretical neutral axis position in table 6 because of slip. In summary, the elastic-plastic calculation method can be used to evaluate the ultimate bending capacity of GFRP-concrete-steel composite beam.

CONCLUSIONS

Based on the experimental results of GFRP-concrete-steel composite beam and the theoretical analysis, the following conclusions are drawn:
1) The failure mode of the GFRP-concrete-steel composite beams (with or without wet bonding technique) were concrete crushed. After the top concrete began to spall, the composite beams could bear higher load because of the confining effect which was provided by GFRP.
2) The flexural stiffness of GFRP-concrete-steel composite beam was lower than that of steel-concrete composite.
beam. Both the GFRP-concrete-steel composite beams with wet bonding interface and unbonded interface at the ultimate load showed great deformation ability and had good ductility.

(3) The slip distribution of GFRP-concrete-steel composite beam was similar to steel-concrete composite beam. The minimum slip position was at mid-span. The values of slip increased from the mid-span and decreased slightly near support.

(4) The elastic-plastic calculation method was suitable for predicting the ultimate bending capacity of GFRP-concrete-steel composite beam.

ACKNOWLEDGMENTS

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ABSTRACT

In this paper, a novel composite bridge deck with prestressed basalt FRP (BFRP) shell and concrete was proposed to solve the excessively low utilization efficiency of FRP shell, as well as the deficient bonding behavior between shell and concrete, in composite bridge deck. The cross-section of FRP shell was firstly optimized by finite element (FE) method. The surface of FRP shell was enhanced by corrugated teeth and adhering sand processing. The experiment on the FRP shells and the composite bridge decks with FRP shell and concrete were conducted under the construction load and ultimate load, in order to validate their effectiveness. The results show that corrugated cross-section with two arches achieves the highest geometric stiffness by unit geometry. The deflection of FRP shell under external loads can be significantly reduced by about 50%, through the camber generated by the prestressing FRP laminates. The composite bridge decks with proposed FRP shell and concrete perform higher stiffness and larger capacity compared to the control specimens.

KEYWORDS

Basalt fiber reinforced polymer (BFRP) shell, composite bridge deck, optimization, prestress, utilization efficiency, static behavior.

INTRODUCTION

Composite bridge deck, making the most of the compressive property of concrete and tensile strength of FRP, is considered as a type of highly efficient structures (Deskovic et al. 1995). The FRP-concrete composite bridge deck was firstly developed for replacing RC bridge deck of Salem Avenue Bridge in America, using pultruded GFRP slab bonded with GFRP tubes as the shell (Lopez-Anido et al. 1999). Although the composite bridge deck performed satisfactory mechanical properties, its capacity was limited due to the poor bond between GFRP slab and tube. Some improvement measures were made, for example, stiffening ribs installed on FRP slab (Cheng et al. 2005). However, those measures mostly caused the difficulty in manufacture and decreased the construction efficiency. Another problem lies in the traditional FRP shell for bridge deck is the low utilization efficiency caused by the stiffness requirement. Some measures such as FRP wrapped steel tubes or pultruded rectangular FRP tubes bonded on FRP slab (Kubo et al. 2001, Hayes et al. 2006) can increase stiffness, but they even lowered the utilization efficiency. To solve that problem, a GFRP shell, with open cross-section and corrugated profile was proposed (Zuo et al. 2014). However, this type of FRP shell still needs a further enhancement from both geometric and structural aspects. In this study, BFRP, with higher modulus, was adopted to replace conventional GFRP. The geometry of the cross-section of FRP shell was optimized to improve stiffness. A prestressed BFRP shell is proposed to offset the deflection under construction load. Furthermore, the static loading tests on the FRP shells and the composite bridge decks were conducted to validate their effectiveness.

OPTIMIZATION OF BFRP SHELL

Modelling

To optimize the cross-section of BFRP shell, a finite element (FE) model was established using ANSYS. Element type of SHELL 63 was adopted. Three alternative types of cross-section are shown in Figure 1. According to JTG D60 (2004), the total longitudinal length of shell was 3000 mm, and the span (L) was 2500 mm. The height and section width of cross-section of BFRP shell were determined to be 150 mm and 250 mm. The dead load, equal to 3.75 kN/m² multiplied by the corresponding partial factor 1.2 (JTG/T F50, 2011), was adopted in FE method.
Results of FE Analysis

According to the results of FE analysis, the deflection of Type 1 equals to 8 mm, exceeding the maximum deflection limitation of shell (L/400) according to GB 50010 (2010), which is unacceptable. Type 2 and Type 3 perform significantly smaller deflections than Type 1. Furthermore, Type 3 shows 18% less deflection than Type 2, although its cross-sectional area is 6% larger than Type 2. Based on the cross-sectional shape of Type 3, the arch radius was also optimized through FE analysis. The deflections of FRP shells, with arch radiiues of 20, 30, 40 and 50 mm, were analyzed by ANSYS. Although the deflection decreases as arch radius increases, the unit geometric stiffness, calculated by Eq. 1, reaches the maximum when the radius equals to 40 mm. Thus, the cross section of Type 3 with arch radius of 40 mm was adopted in the following experiment.

\[ s = \frac{1}{f \times A} \]  

(1)

in which, \( s \) = unit geometric stiffness; \( f \) = deflection; \( A \) = cross-sectional area.

The optimized cross-section shape of BFRP shell is shown in Figure 2. To enhance the connection between FRP shells, corrugated teeth were designed to distribute along the height direction of the web.

EXPERIMENTAL PROGRAM

Preparation of BFRP Shell Component

BFRP shell components adopted in this study were manufactured by using basalt fiber roving and unsaturated polyester resin through pultrusion, with a specially designed steel mold. Bi-directional basalt fiber sheets were also adopted during pultrusion to enhance the transversal mechanical properties. The tensile strength and elastic modulus of the BFRP shell are 622 MPa and 31 GPa according to previous tests. Three BFRP shell components were assembled into a whole BFRP shell structure in the following loading experiments, to simulate the practical engineering application of composite bridge deck. The reliable connection between FRP shell components was realized through local bolts, corrugated teeth and Sikadur-30 sticky adhesive.

Prestressed BFRP Shell

Prestressing technique was adopted to offset the deflection under construction load and further enhance the stiffness of FRP shell. Prestressing BFRP laminates (tensile strength: 775 MPa, elastic modulus: 35 GPa), bonded on the inner surface of lower flange, was used (Figure 3). The BFRP laminates, with similar mechanical properties to the BFRP shell, were adopted to avoid excessively large shear stress on their interfaces. Sikadur-30 sticky adhesive was gelatinized between the laminates and shell.
Tensioning of Prestressing BFRP Laminate

The specific tensioning setup is shown in Figure 4. At the tensioning end, the tensioning force was applied by a hydraulic jack through a tension rod. A hollow steel plate was used to transfer load during tensioning. Three adjustable bolts were used to ensure the synchronicity of BFRP laminates. BFRP laminates were all anchored by steel clamps (200 mm long) with bolts through friction. Two stiff steel plates at both ends were adopted to balance tensioning force on the BFRP shell through.

![Figure 4 Tensioning apparatus](image)

Before tensioning, the bonding surfaces on the BFRP laminates and shell were polished to be rough. The adhesive was then gelatinized uniformly along the bonding area. After the assembly of the tensioning apparatus, the prestress was firstly applied to 2 kN, and the bolts were adjusted to make the loads in three laminates be equal. Then, the laminates were tensioned, with 10 kN per step, to the predetermined tensioning force (40 kN). After tensioning, the stresses in BFRP laminates were equal to about 0.42 fu, less than the creep rupture stress of BFRP (0.52 fu) (Wang et al. 2014). The load was sustained for 48 h at a constant temperature of 30 degrees centigrade, to ensure sufficient strength of the adhesive achieves. Figure 5 shows the FRP shell during and after prestress tensioning.

![Figure 5 Prestressed BFRP shell (a) during prestress tensioning (b) after prestress tensioning](image)

**Simulation of Construction Load**

A sand casting experiment was conducted to simulate the construction process of concrete casting, due to the convenience in handling (Figure 6). The action of sand on FRP shell is similar with that of concrete during construction. According to JTG/T F50 (2011), the load of sand for the checking calculation of stiffness and strength were respectively calculated to be 10.8 kN and 25 kN, respectively. Timber frameworks were erected around the FRP shells. Data were recorded every 0.5 kN of sand loading. The span of BFRP shells was 2500 mm.

**Static Behavior Test on Composite Bridge Decks**

*Preparation of specimens*

Four composite bridge deck specimens were manufactured for the static behavior test, which simulates the state of bridge deck under vehicle load. BFRP rebars were adopted as the stirrups and longitudinal reinforcements in the concrete (Figure 7). Concrete with strength grade of C50 was cast into the FRP shells, which were also used as formwork during casting. The experimental parameters included prestress, surface treatment (adhering sand processing in Figure 8) surface profile (corrugated teeth), as shown in Table 1.
Figure 6 Construction loading simulation by sand

Figure 7 Reinforcements

Figure 8 Adhering sand processing

Table 1 Parameters of specimens

<table>
<thead>
<tr>
<th>Number of specimen</th>
<th>Corrugated teeth</th>
<th>Adhering sand processing</th>
<th>Prestress</th>
</tr>
</thead>
<tbody>
<tr>
<td>BD-1</td>
<td>Y</td>
<td>Y</td>
<td>N</td>
</tr>
<tr>
<td>BD-2</td>
<td>Y</td>
<td>N</td>
<td>N</td>
</tr>
<tr>
<td>BD-3</td>
<td>N</td>
<td>N</td>
<td>N</td>
</tr>
<tr>
<td>BD-4</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
</tr>
</tbody>
</table>

Loading procedure

The span of each specimen was determined to be 2500 mm. Three-point loading was adopted through two synchronous actuators (Figure 9), with the total loading capacity equal to 1000 kN. A steel plate, with length of 600 mm, width of 250 mm and thickness of 50 mm, was positioned at the mid-span to simulate wheel-pressure. The load was applied with the speed of 10 kN per step until 100 kN was reached. Then the load was applied with 5 kN per step. The data of mid-span deflection and load were collected by computer during the test.

RESULTS AND DISCUSSIONS

Deflections of Non-Prestressed and Prestressed BFRP Shells under Construction Load

The load-deflection curves of non-prestressed and prestressed BFRP shells under construction load are shown in Figure 10. At stiffness checking calculation load (10.8 kN), the mean deflection of non-prestressed shells in the mid-span is 4.0 mm, consistent with that calculated by FE method (4.16 mm). The deflections of prestressed BFRP shells are almost zero. Under strength checking calculation load (25 kN), the maximum deflection of prestressed FRP shell is only 3.8 mm, 50% lower than that in non-prestressed BFRP shell, showing the advantage of camber caused by prestress.
Figure 10 Construction load-deflection curves of non-prestressed and prestressed FRP shell

**Static Behavior of the Composite Bridge Deck**

The values of ultimate load and deflection under ultimate load, and failure modes are listed in Table 2. The load-deflection curves of each specimen are shown in Figure 11. The effect of adhering sand processing on enhancing the bond behavior between concrete and FRP shell can be apparently observed from the failure mods of BD-1 and BD-2. Furthermore, compared with BD-2, BD-3 performs lower stiffness and earlier slippage between concrete and FRP shell due to its lack of corrugated teeth. Finally, from the comparison between BD-1 and BD-4, prestress has no obvious effect on the ultimate load, since the failure modes of the two specimens were both dominated by concrete crushing. In conclusion, the proposed composite bridge deck possesses advantageous static behavior.

<table>
<thead>
<tr>
<th>Number of specimen</th>
<th>Ultimate load $F_u$ (kN)</th>
<th>Deflection under $F_u$ (mm)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>BD-1</td>
<td>618</td>
<td>30.2</td>
<td>Concrete crushing, no C-S slippage*</td>
</tr>
<tr>
<td>BD-2</td>
<td>395</td>
<td>34.1</td>
<td>C-S slippage</td>
</tr>
<tr>
<td>BD-3</td>
<td>412</td>
<td>42.9</td>
<td>C-S slippage</td>
</tr>
<tr>
<td>BD-4</td>
<td>619</td>
<td>28.6</td>
<td>Concrete crushing, no C-S slippage</td>
</tr>
</tbody>
</table>

* C-S slippage is short for the slippage between concrete and FRP shell.

Figure 11 Load-deflection curves of (a) BD-1; (b) BD-2; (c) BD-3; (d) BD-4

**CONCLUSIONS**

A novel composite bridge deck with FRP shell and concrete was proposed in this paper. FE method was used to optimize the cross-section. Prestress was applied onto FRP shell through FRP laminates to enhance the utilization efficiency of FRP shell. Construction loading and static behavior tests were conducted on the proposed FRP shell and composite bridge deck. The main conclusions are as follows.

1. Through FE method, the cross section with two arches (radius equal to 40 mm) was optimized for FRP shell, which achieves the highest unit geometric stiffness among the three candidates.
(2) Prestress can generate camber and reduce the deflection of FRP shell under construction load by 50%. The utilization efficiency of FRP shell is prospective to be further enhanced.

(3) Corrugated teeth and adhering sand processing of FRP shell surface can significantly enhance the connection between FRP shells and bond behavior between FRP shell and concrete, respectively, and thereby increase the load capacity and stiffness of composite bridge deck.

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ENERGY DISSIPATION CHARACTERISTICS OF CONNECTIONS FOR HYBRID AND FRP INFILL PLATES FOR SHEAR WALLS

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ABSTRACT:
The latest developments in steel shear walls (SSW) are a clear indication about the effectiveness and practical applicability of systems consisting of steel frames and thin infill plates. One of the main factors defining the behaviour of such systems is the type of connection between the frame and the infill plate. The estimation of this factor is getting more complicated in case of recently developed hybrid and FRP infill plates. This paper assesses ways of improving energy dissipation properties of steel to hybrid (steel/GFRP) and steel to plain GFRP connections. Different combinations of connections between steel hybrid and FRP plates were developed and tested with the aim to achieve higher energy dissipation. The size of all the plates was 125mmX70mm, with the connection to the steel plates via two M8 bolts. Variations included the application of additional layers of GFRP at the zone of the bolted area, use of adhesive film and additional epoxy around the bolted area. Both ultimate load capacity and energy dissipation of the tested specimens were estimated and analysed. Conclusions leading to the use of the most effective types of connections are offered.

KEYWORDS
Shear walls, GFRP, connections, load capacity, energy dissipation

INTRODUCTION:
This paper is part of a bigger project focused on assessing and improving the energy dissipation of hybrid steel/FRP shear walls. Connections between the infill panel and the outer frame are one of the main factors influencing energy dissipation. Due to the complexity and relatively new application of the hybrid infill plates, detailed assessment must be undertaken to find out the most efficient connecting system in aspect of increasing energy dissipation of the hybrid shear walls.

There has been some research assessing the modes of failure and ultimate capacity of FRP connections. However, there is very limited research on improving energy dissipative properties of FRP and Hybrid steel/FRP connections. Such type of research is crucial in the long term development and application of the innovative FRP and hybrid steel shear wall (SSW) structural systems.

BACKGROUND
The energy dissipation in hybrid shear walls is developed in three different areas; the steel frame, the hybrid infill plate and the connection between them.

Some of the main factors that contribute to the energy dissipation of steel and hybrid steel/FRP shear walls are; plate aspect ratio (span/height), thickness of infill plate, type of column to beam connection, the connection between the infill plate and the outer frame elements as well as the application of different types and orientation of FRP’s. Cut outs in infill shear panels is another factor that influences energy dissipation and ultimate load capacity, as assessed by Maleki (2010). He concluded that the stiffness, energy dissipation and load carrying capacity decrease for specimens with cut-outs. Maleki added that this issue could be resolved via the application of additional stiffeners.
The connection of the infill plate to the exterior frame is one of the main contributors to energy dissipation. Choi and Park (2008), tested the effect of application of various details of SSW infill plate connection to the boundary frame elements. Two of them were M20 bolted connections at spacing of 100mm with a 6mm fish plate and two sided fillet welded connections to the same thickness fish plate. They determined that the effects of bolted connections and welded connections with respect to energy dissipation were approximate the same, up to a drift of 3.6% and that bolted connections exhibited similar initial stiffness and slightly higher load carrying capacity. However, the ratio of cumulative energy dissipation capacity of the bolted connections to the welded connections from their research was 0.52.

FRP structural connections have various failure modes in plane of loading some of the most common are; net section tension failure, shear out failure, bearing, cleavage, slip failure and possibly a combination of the mentioned modes (Zhou and Zhao, 2013). Turvey, G (2000) also confirmed in his overview that for pultruded FRP structural connections with single bolt fastening and an axial load at an angle of 0°, 45° and 90° to the pultruson, there are four basic modes of failure and they are: bearing, tension, shear out, and cleavage. Turvey added that, when the ratios of the end distance to the diameter of the bolt and the end distance to width of specimens are high the failure mode is bearing. Rosner and Rizkalla (1995) developed a design procedure based on their research assessing the behaviour of bolted fibre reinforced polymer connections, taking into account the materials orthotropic properties and pseudo yielding behaviour. They obtained failure envelopes for fibre orientation of 0°, 45° & 90°, depending on the ratios of the hole diameter to width (d/w) and edge distance to hole diameter (e/d). It was concluded that for the connections with a ratio of d/w & e/d falling on the part of the envelope that is curved the failure mode is net tension and for the straight part of the envelope the failure will be bearing or cleavage.

Manalo et al. (2008) analysed the behaviour of hybrid GFRP/CFRP bolted connections both with adhesive and without. It was evaluated that the connections without adhesive failed via bearing of bolts and crushing of fibres, while for the connections with adhesive resulted initially in bearing and delamination between the CFRP and GFRP and eventually net tension of the connection at failure load. Manalo also assessed the effect of bolt torque, revealing that there is a slight increase in capacity with increase of torque.

Research by Petkune et al. (2014) investigated the effects of variation in the type of hybrid and FRP plates on the capacity of plate-to-steel element connections. Hybrid plates with CFRP prepreg unidirectional fabric attached to steel plate and CFRP only laminated plates were investigated as well as application of additional adhesive around the bolted area. It was determined that there is a significant increase in capacity when utilising adhesive around the bolted area. The presented research is focusing mainly on the improvement of energy dissipation in the connections between the frame and infill late.

**METHODOLOGY:**

**Specimens:**

Numerous specimens with a width of 70mm and a length of 125mm were tested. Three samples were tested for each set of specimens and the average results analysed. Varying factors were: the application of additional 2 layers of E722-02 UGE400-02 32%rw (GFRP) at a {−45/+45} orientation on either side of bolted area, application of DP110 Scotch-Weld adhesive and the use of EF72 adhesive film from TENCATE. The adhesive film was applied between the steel layer and the GFRP, as a solution to delamination issues for hybrid specimens. Figure 1 & Table 1 give information about of the geometry, type and number of samples.

The orientation of the specimens with 8 layers of GFRP pre-preg plies were applied at a {−45/+45−45/+45/+45−45/+45−45} orientation. For the hybrid specimens incorporating steel plates the orientation is {−45/+45/S/+45−45}. The test specimens were tested using a displacement control load at an applied rate of 2mm/min. The applied displacement was continued after reaching maximum load capacity to fully assess the mode of destruction and energy dissipation of the specimens.
Table 1 Specimens

<table>
<thead>
<tr>
<th>Number of Bolts</th>
<th>0.8m steel plate (S)</th>
<th>GFRP (G)</th>
<th># of layers of GFRP</th>
<th>Additional 4 layers of GRP in bolted area (F)</th>
<th>EF72 adhesive film (E)</th>
<th>DP110 Epoxy (D)</th>
<th>Average thickness t1(mm)</th>
<th>Average thickness t2(mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2S</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>2G8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2.13</td>
<td>2.13</td>
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<tr>
<td>2G8D</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3.05</td>
<td>2.13</td>
</tr>
<tr>
<td>2G8F</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3.05</td>
<td>2.12</td>
</tr>
<tr>
<td>2G8FD</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>3.05</td>
<td>2.12</td>
</tr>
<tr>
<td>2G4</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>3.05</td>
<td>2.13</td>
</tr>
<tr>
<td>2G4F</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3.05</td>
<td>2.13</td>
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<tr>
<td>2G4FE</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>3.05</td>
<td>2.13</td>
</tr>
<tr>
<td>2G4FED</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3.05</td>
<td>2.13</td>
</tr>
</tbody>
</table>

**Preparation:**

Application of adhesive film EF72 was used between the GFRP and the steel plate to increase the bond between the two layers. Before the film was applied the steel plate was abraded using sand paper and cleaned using acetone. Further to the application of the GFRP, the specimens were placed in a sealed vacuum bag between two plates attached to a 1 bar vacuum pump and placed in an oven for 1hour at 120 °C according to curing specifications of the manufacturer. Images of these steps are illustrated in Figure2.

**RESULTS & ANALYSIS:**

The main factors investigated were the load capacity and the energy dissipation. The energy dissipation was presented by the enclosed area under the load displacement graphs.
For better comparison between the samples they were divided into three groups. The first group included all the specimens consisting of 8 layers of GFRP (2G8, 2G8D, 2G8F and 2G8FD), the second group comprised of the hybrid steel/GFRP samples with 4 layers of GFRP and a 0.8mm steel plate in between them (2SG4, 2SG4F and 2SG4FD) and the third group consisted of hybrid specimens that employed EF72 adhesive film between the steel plate and the first GFRP ply (2SG4FE and 2SG4FED). The behaviour of the samples from each of the three groups is displayed in graphs in Figures 3-5.

Figure 3 illustrates all the specimens that incorporate eight layers of GFRP. The varying factors in the group consisted of the application of Dp110 epoxy around the bolted area (2G8D), the use of additional GFRP strips around the bolted area (2G8F) and the application of both epoxy and additional GFRP (2G8FD). The application of either the adhesive or the additional layer of GFRP shows that the ultimate load capacity for both 2G8D & 2G8F specimens was increased significantly by 103% and 90% respectively when compared to 2G8. However 2G8D exhibited complete failure at approximately 16mm displacement resulting in an increase of 106% in the energy dissipation, while 2G8F displayed gradual destruction allowing it to obtain slightly higher energy dissipation of 121% when compared to 2G8. When applying both additional layers of GFRP and epoxy, the specimen showed an increase in capacity from 8.73KN for 2G8 to 21.35KN.

The second group contained of hybrid steel/GFRP specimens with 0.8mm steel plate and 4 layers of GFRP presented in Figure 4. The variations were of the application of additional GFRP strips around the bolted area (2SG4F) and the application of both adhesive and additional GFRP strips around the same area (2SG4FD). For this group increase in load capacity and energy dissipation was not as drastic as for the pure GFRP specimens in the first group. For the reference hybrid specimen (2SG4) the maximum load and energy dissipation achieved was 12.98KN and 146.08KN.mm. When comparing 2SG4F to 2SG4 the ultimate load capacity was increased by 41% and energy dissipation increase by and 29% for 2SG4F. Applying both additional layers of GFRP and epoxy (2SG4FD) resulted in an increase in maximum load capacity to 24KN and increasing the energy dissipation to 276.5KN.mm.

The graph in Figure 5 compares the hybrid steel/GFRP plates that have additional adhesive film between the steel plate and the GFRP layer as well as additional GFRP around the bolted area (2SG4FE) with the plate that has adhesive film between the steel plate and the initial GFRP layer and both additional adhesive and GFRP around the bolted area (2SG4FED). 2SG4FED shows a clear increase of approximately 37% in the load capacity of the specimen and a 52% rise in energy dissipation when comparing it to 2SG4FE.
Figure 4 Load Vs Displacement of Hybrid connections without EF72 adhesive film

Figure 5 Load Vs Displacement for specimens with adhesive film between the steel plate and the initial GFRP layer

Figure 6 Energy Dissipation & Maximum Achieved Load

Figure 6 is allowing comparison of the maximum load capacity and the energy dissipation of all specimens. The application of epoxy around the bolted area significantly increases the load capacity due to the reduction in slippage between the specimen and the clamping steel plates. Applying additional FRP layers around the bolted area increases the bearing capacity of the specimen due to increased thickness of FRP around the bolts. The application
of adhesive film in hybrid specimens has a minimal effect on both load capacity and energy dissipation, visible when comparing 2SG4F with 2SG4FE and 2SG4FD with 2SG4FED. This minimal effect of additional adhesive film is applicable only for the connections and the effect is not valid for other areas of the infill plate. It was indicated by Petkune et al. (2014) that the application of adhesive film between the steel and CFRP prepreg laminates over the whole infill plate can significantly improve the shear wall behaviour.

While assessing the failure modes it was revealed the most common modes of failure for connection with two bolts were shear-out, cleavage failure and a combination of both. Where adhesive was used to prevent slippage at initial stages of loading, in some specimens failure occurred in mid-section of the specimen resulting in higher load capacity. Some of the specimen’s failure modes can be seen in Figure 7.

![Figure 7 Modes of failure starting from the left; (I) sample before testing, (II) shear out, (III) cleavage, (IV) combination of both shear out and cleavage and (V) failure at mid-section of plate.](image)

**CONCLUSION**

From testing and analysing different variations of hybrid steel/GFRP connections the following conclusions can be made:

1- For GFRP only plates application of either additional layer of GFRP or adhesive around the bolted area will increase both the ultimate capacity and the energy dissipation (Figure 3)

2- Simultaneous application of additional GFRP strips and adhesive around the bolted area will result in even higher ultimate load capacity and similar increase of the level of energy dissipation (Figure 3)

3- For hybrid plates the result of applying additional layer of GFRP and adhesive is similar to the results for plain FRP plates when both materials are applied simultaneously, with significant improvement of the ultimate load capacity and energy dissipation (Figure 4)

4- Additional epoxy layer between the steel plate and the adjacent GFRP layers at the zone of bolting is not so beneficial in increasing the capacity and energy dissipation as the application in internal zones of the infill plate.

5- Comparison of the obtained results with steel only connection indicates that in most of the cases the capacity and energy dissipation for the developed connections are significantly higher up to 2.9 and 2.8 times respectively.

**REFERENCES:**


BEHAVIOUR OF STEEL FRAMED SHEAR WALLS WITH FRP INFILL PLATES

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\textsuperscript{1}Faculty of Science, Engineering and Computing, Kingston University London, UK

ABSTRACT

Excellent properties of FRP materials are making them attractive component for developing new structural elements and systems. Initial investigations indicated positive effect of using hybrid (steel/FRP) infill plates on the ultimate capacity and energy dissipation capacity of steel framed shear walls. However, the usage of pure FRP plates has not been sufficiently investigated till the moment. The proposed research is investigating the behaviour of steel framed shear walls with GFRP and CFRP infill plates via experimental testing of medium scale samples. The testing of specimens has been conducted via application of displacement controlled quasi-static loading following ATC-24 protocol. Obtained results are analysed and compared with the results for steel framed shear walls with steel infill plates. Corresponding conclusions and considerations for possible future applications are offered.

KEYWORDS
FRP, shear wall, composite plate, quasi-static loading.

INTRODUCTION

The behaviour of steel shear walls was investigated for several decades (Roberts, 1995; Astaneh, 2001). They were used in the number of high rise buildings as primary lateral loading system. Steel shear walls have significant advantages in terms of light weight, high stiffness and energy absorption in comparison to other structural systems. Hybrid structural elements are becoming popular in thin walled structures, sandwich panels and shear walls. Hybrid shear walls (HSW) are innovative structural lateral load resisting system, HSW are defined to be consisted of steel framed elements and steel infill plates laminated with fibre reinforced polymers (FRP) material. By reducing out-of-plane deformations of the infill plate and increasing energy absorption capacity, such systems have the potential to provide highly effective structural solution (Rahai et al., 2011; Maleki et al. 2012; Petkune et al., 2014). Based on the previous studies it was found that the most effective angle of fibers inclination is in the direction of diagonal tension field action.

This study investigates the use of the innovative pure FRP infill plates within the steel boundary elements and compares behaviour and results to the steel shear wall with steel infill plate only. Glass FRP (GFRP) and carbon FRP (CFRP) were used to produce composite infill plates. Fibers were placed at 45° inclination angle, which corresponded to the angle of diagonal tension field action.

EXPERIMENTAL SET-UP

Specimen Description

Single-storey medium scale specimens were tested in the project. Shear wall scaled models, consisting of steel frame and infill plate, were designed at Kingston University London. The steel frames tested in this study were 1/3 scaled model of a shear wall with a height of 1025 mm and width of 1090 mm. The steel frame members consisted of two columns and beam made from Universal Beam Section 127 x 76 x 13 mm (S355 grade) shown in Figure 1a. They were fabricated, assembled and welded by Cannon Steels Ltd. Primary fish plates were welded continuously to the internal part of the steel frame. Three different types of infill plates were used: steel, carbon FRP (CFRP) and glass FRP (GFRP). Steel shear wall specimen (SSW) was designed with steel infill plate 0.8 mm thick (S275 grade). For comparability shear walls with CFRP (CSW) and GFRP (GSW) had the same design configuration, they were made with eight layers of prepreg FRP material and fibres were placed at 45° inclination. Shear walls with CFRP and GFRP plates were referred as composite later in the text. Specification of the specimens is shown in Table 1.
Figure 1 a) Dimensions of single-storey shear wall specimen b) Testing Rig

### Table 1 Specimen description and characteristics

<table>
<thead>
<tr>
<th>Name of the specimen</th>
<th>Design configuration of the infill plate</th>
<th>Weight of the infill plate, kg</th>
<th>Thicknesses of the infill plate, mm</th>
<th>Connection between infill plate and fish plates</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Shear Wall (SSW)</td>
<td>Steel [S]</td>
<td>4.08</td>
<td>0.80</td>
<td>Bolted</td>
</tr>
<tr>
<td>Carbon FRP Shear Wall (CSW)</td>
<td>[-45/+45/-45/+45/-45/+45/-45/+45/-45]</td>
<td>2.24</td>
<td>1.68</td>
<td>Bolted + adhesive</td>
</tr>
<tr>
<td>Glass FRP Shear Wall (GSW)</td>
<td>[-45/+45/-45/+45/-45/+45/-45/-45+]</td>
<td>3.26</td>
<td>2.52</td>
<td>Bolted + adhesive</td>
</tr>
</tbody>
</table>

NOTE: S- steel; -45/+45- angle of the fibres inclination

For CSW specimen Unidirectional CFRP type MTM 28-1 series prepreg produced by Cytec Industrial Material Ltd was used. For GSW specimen Unidirectional GFRP prepreg E722-02 produced by TenCate Advanced Composites Ltd was used. Mechanical properties of these materials found in accordance to ASTM D3039/D3039M-14 and ASTM D3518/D3518M-94 are shown in Table 2.

### Table 2 Properties of the FRP materials used

<table>
<thead>
<tr>
<th></th>
<th>Unidirectional CFRP type MTM 28-1 series prepreg</th>
<th>Unidirectional GFRP prepreg E722-02</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s Modulus E11, GPa</td>
<td>140</td>
<td>40.95</td>
</tr>
<tr>
<td>Young’s Modulus E22, GPa</td>
<td>8.5</td>
<td>10.5</td>
</tr>
<tr>
<td>Shear Modulus G12, GPa</td>
<td>5.8</td>
<td>3.3</td>
</tr>
<tr>
<td>Poisson’s ratio V12</td>
<td>0.319</td>
<td>0.311</td>
</tr>
</tbody>
</table>

The composite plates were prepared with oven/vacuum method. Prepreg FRP layers were placed by layering them between two steel plates used as moulds. First layer of the FRP material was rolled on the cover steel plate mould, then the rest were laid in accordance to design specifications with fibre orientations at ±45° angle of fibres, eight layers in total. A vacuum breather cloth was wrapped around the specimen, then the specimen was sealed, vacuum bagged and cured inside an oven under vacuum. The curing temperature increased at a rate of 3°C per minute until 120°C and an even pressure up to 980 mbar was applied by using a vacuum pump. Then the temperature was kept constant at 120°C for 1 hour and finally the temperature decreased to 60°C during the cooling down cycle and the sample was then left to cool to room temperature outside the oven.

**Connection Specification**

Two types of the connections were used during testing: bolted connection and bolted connection with adhesive. For steel shear wall specimen a traditional bolted connection was used. M8 bolts are placed sat 70 mm spacing centre-to-centre. They had a minimum breaking torque of 32 N-m for A4-70 stainless steel class, where “A” means cold worked austenitic stainless steel and “70” represents a minimum tensile strength of 700 N/mm².

For composite specimens bolted with adhesive connections were used, which proved to be effective as a result of the previous studies (Petkune et al., 2014b). It was concluded that the use of the adhesive improves the capacity
of the connections by 20%. Devcon epoxy plus adhesive (manufactured by ITW Polymers Adhesive) with shear strength of 20 MPa was used. The adhesive was applied evenly on edges of infill plate and primary fish plates, the composite surfaces were connected and were bolted together with M8 bolts to the specified torque. Any excessive adhesive coming out from the edges was removed. To achieve good bond at the connection, specimen was cured at room temperature for at least 24 hours before testing. In order to provide a stiff connection to the composite infill plate, an additional steel strip 30 mm wide and 1.5 mm thick was attached to both sides around the edges of the infill plate using the same adhesive. Holes were drilled after the strips were attached.

Table 3 Connection between fish plates and infill plates

<table>
<thead>
<tr>
<th>Connection</th>
<th>Scheme for connections between infill plate and fish plate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolted connection</td>
<td>![Diagram of bolted connection]</td>
</tr>
<tr>
<td>Bolted and adhesive + additional steel strips</td>
<td>![Diagram of bolted and adhesive connection with additional steel strips]</td>
</tr>
</tbody>
</table>

**TESTING METHOD**

Figure 1b shows testing rig used for testing specimens in accordance to ATC-24 protocol (1992). Specimen was fixed to the reaction frame and lateral supports prevented out-of-plane movement of the steel frame. Controlled displacement to the specimen was applied via a screw jack and measured with load cell. Displacement was measured with LVDTs. Figure 2 shows a cyclic sinusoidal loading and applied to all specimens for a range of different amplitudes varying from 0.4 mm to 35 mm displacement. The application of the same displacement in both directions completes one cycle of loading. Initially, three cycles at each amplitude were applied, and then above 15 mm displacement the number of cycles was decreased to two cycles per amplitude according to the ATC-24 protocol. The loading is designed for steel shear wall samples and is kept the same for FRP samples with the aim to achieve comparability.

![Loading Protocol](Figure 2 Cycling loading in accordance to ATC-24 protocol)

**RESULTS AND DISCUSSIONS**

**Behaviour of the specimens during the testing**

The steel shear wall specimen SSW was loaded up to 35 mm displacement. Until displacement of 3.5 mm, diagonal tension action was with elastic deformations and the initial shape of the plate was recovering in the end of the cycles. However, when the infill plate had undergone plastic deformations for displacements above 3.5 mm, out-of-plane deformations were more visible and initial shape was not recovering at the end of the cycle. At
displacements higher than 10 mm, enlargement of holes around bolts in the connections between fish plates and infill plates started. It led to the yielding of the steel infill plate and its tearing around bolt holes, corresponding noise occurred when tearing started. Sliding of the infill plate progressed with the increase of the displacements. After the 15 mm displacement cycle, up to 4 mm sliding from the original position of the border of the infill plate at the top part of the wall was noticed. The initial pinching of the infill plate along diagonal tension field started at a displacement of 15 mm, which further progressed to development of small holes at displacements higher than 30 mm. Development of the plastic hinges at the bottom of the columns of the steel frame was noticed at displacements above 15 mm, and appeared at the top of the columns above 30 mm displacement. At 35 mm displacement, sliding of the plate reached 15 mm at the top border between fish plate and infill plate, and the extension of the bolt holes were visible at fish plate section. Due to the concentrations of stresses in corners of the infill plate, steel snapped forming diagonal cracks at top corners. The final damage of the steel shear wall specimen (Figure 3a) occurred through further development of the plastic hinges around the bottom and top areas of the column and tearing of the steel plate around bolt holes. Test was terminated when significant level of the damage in the infill plate and the frame occurred and the design range of deformations for testing equipment was achieved.

Figure 3 a) SSW specimen after 35 mm displacement b) CSW specimen tested to 25 mm displacement: opening of the crack, ruptures in the specimen, delamination of thin strips of CFRP

CSW specimen was tested until exceeding the ultimate load value which occurred at 20 mm displacement. For the CSW specimen diagonal tension field action through the development of out-of-plane deformations was visible at 2.5 mm displacement. The diagonal waves recovered at the end of loading cycle up to the displacement of 5 mm. Above 5 mm residual deformations were recorded. Tearing and appearance of the first diagonal cracks of the CFRP layers in the corners were developed when specimen was tested to the displacement higher than 10 mm. These cracks in FRP fabric increased longitudinally in diagonal direction at higher displacements. When displacements increased to 15 mm and higher, thin strips of the CFRP perpendicular to the cracks started to delaminate in top corners of the infill plate. At 20 mm displacement, a crack through the whole thickness of the CFRP plate developed. The layer of the adhesive at the connection between fish plates and infill plate cracked when the specimen was tested to 20 mm displacement, however no visible movement of the plate at the connections was recorded. Development of the plastic hinges was noticed at 20 mm displacement in the steel boundary elements. Maximum load of 266 kN was achieved at 20 mm displacement. In contrast to SSW specimen which experienced plastic deformations of the plate and connections, carbon specimen failed at early stages of loading due to the lack of yielding process with significant cracks developing in the direction of diagonal tension field (Figure 3b).

GSW specimen was tested until 35 mm displacement. For GSW specimen after 2.5 mm displacement, out-of-plane buckling of the plate was not fully recovered at the end of the cycle. At 15 mm displacement, crack in the layer of adhesive at the connections was observed near top section of the plate. At 20 mm displacement delamination and tearing of the FRP in corners of the infill plate started and development of the plastic hinges occurred at the bottom of the shear wall columns. The glass FRP infill plate was torn in one of the corners with about 40 mm long crack in FRP layers. At 25 mm displacement, maximum load of 359 kN was achieved. At 30 mm displacement, further delamination of the GFRP developed in both corners of the infill plate. For GSW specimen cracks did not go through all layers of the GFRP fabric, as they were developed for the CSW specimen. At 30 mm displacement, the infill plate slid and a significant gap developed between infill plate and fish plate in adhesive layer in the corners of the infill plate. Due to the diagonal tension field action, fish plates buckled. With the increase of the displacement to 35 mm (Figure 4), in the GSW specimen, areas of the delamination were widened in the corners. Further sliding of the plate occurred and elongated bolt holes were visible, which happened at a later stage in comparison to other specimens. At 35 mm displacement signs of local buckling in the columns of the frame at the zones of plastic hinges was detected.
Hysteresis Curves and Load vs Displacement and Energy Absorption graphs

Based on the hysteresis curves shown in Figure 5, load vs displacement (Figure 6) was made. Loads were calculated by taking the average from the extreme values of the cycles at the same displacement amplitude.

The initial load values for the same levels of displacement for composite specimens were lower than the control specimen SSW. Above 6.5 mm displacement for GSW specimen and above 9 mm displacement for CSW specimen, both specimens showed better load values than the control specimen. Up to 11 mm displacement, GSW and CSW behave similarly, but above this level the difference between the two specimens starts to increase significantly. The load capacity of GSW specimen is higher than capacity of CSW specimen. The difference between two specimens is 20% at 20 mm displacement and increasing up to 33% at 25 mm displacement.

For the CSW specimen the main mode of the failure was a sudden and fast development of delamination and intensive cracking within the CFRP fabric resulting in a relatively brittle failure of the infill plate. In comparison with CSW specimen, GSW had more gradual failure of the infill plate. The connections between the infill plate and the fish plates behaved reasonably well up to the termination of the test. The GSW ultimate load capacity of 359 kN was 35% higher than the CSW ultimate capacity of 266 kN.
The sample with the GFRP infill plate (GSW) has better behaviour and higher values of energy absorption (Figure 7) than the specimen constructed with pure CFRP infill plate (CSW). The energy absorption of the CSW specimen was lower in comparison to other specimen at all levels of loading. Energy absorption of both pure FRP specimens were similar up 15 mm displacement, then the difference between the two specimens increased to 3.0 kJ at 25 mm. The values of energy absorption for the GSW specimen were close to the values of SSW control specimen up to 25 mm displacement.

CONCLUSIONS

The following conclusions can be made:
- The best behaviour in terms of the load capacity and energy absorption is the behaviour of GSW.
- Based on the hysteresis curves, results demonstrated that SSW and GSW specimen have approximately the same energy dissipation capacity up to 25 mm, energy dissipation of the CSW is the lowest.
- CSW is having more brittle failure at early stages of the loading in comparison with GSW.
- The obtained results indicate that GSW specimen could be the successful replacement of SSW.
- Special consideration in design of connections for composite specimens is needed.

ACKNOWLEDGMENTS

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REFERENCES


Performance under Seismic and Severe Loadings
COMPOSITE MATERIALS AS PART OF AN OPTIMAL STRATEGY FOR SEISMIC RETROFITTING OF RC FRAMES

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ABSTRACT
This paper proposes a rational strategy for retrofitting Reinforced Concrete (RC) frame structures, based on combing member- and structure-level techniques, in order to achieve optimal design objectives within the framework of a potentially Multi-Criteria Performance-Based approach. Composite materials, with emphasis on Fiber-Reinforced Polymers (FRP), can be employed in local strengthening interventions, such as confinement of columns, hence representing a viable member-level technique. Therefore, this paper presents the key features of a novel numerical procedure that implements a genetic algorithm capable of selecting the “fittest” solution among the technically feasible ones consisting of alternative configurations of steel bracing systems (as structure-level technique) and FRP-based member-level interventions. The paper reports the main assumptions about the representations of “individuals” as part of this genetic algorithm procedure, along with some details on the algorithm operations (i.e. selection, crossover and mutation). Finally, a preliminary application is proposed.

KEYWORDS
RC frames, seismic retrofitting, soft-computing.

INTRODUCTION
Existing Reinforced Concrete (RC) frame structures were often designed for gravitational loads only, without taking into account the Capacity Design rules introduced by modern seismic design codes (CEN 2005). Therefore, seismic retrofitting of these RC structures is emerging as a technical challenge (fib 2003) and a relevant societal priority, as a result of the significant damage caused by recent earthquakes.

As a matter of principle, retrofitting solutions can generally be grouped into two broad classes that collect, on one hand, the so-called “member-level” (local) techniques and, on the other hand, the structure-level (global) techniques (fib 2003). Techniques belonging to the former, such as confinement of single beams or columns (Rodriguez and Park 1991), aim at improving the capacity of single structural members, in order to contribute to the capacity of the entire structure. The use of composite materials is more and more common in member-level techniques (fib 2006). Conversely, structure-level techniques consist of linking the existing structure with new substructures, such as RC shear walls or steel bracing systems, intended at working in parallel with them. They aim at increasing strength and stiffness of the coupled structural system and, hence, reducing the displacement demand on the existing RC structure and its members (Martinelli et al. 2015).

The aforementioned techniques, considered on their own, represent two somehow “extreme” solutions of the retrofitting problem. However, they may be duly combined with the aim to obtain a synergistic action in both increasing seismic capacity of under-designed members and reducing demand on the existing structure. In fact, each one of the potentially infinite combinations of member- and structure-level interventions leads to different direct costs, life-cycle costs, reliability levels and other quantitative/qualitative parameters that describe the earthquake response of RC structures. In this light, choosing the “optimal” combination of these interventions is clearly an optimization problem, which can be based on the assumption of a properly defined objective functions related to the cost and performance parameters. Although minimising direct costs is often assumed the key objective in retrofitting, a more comprehensive choice of performance parameters (i.e. minimum floor acceleration in presence of sensitive acceleration devices, maximum reliability and so on) may be considered within the framework of a Multi-Criteria optimization problem (Chankong and Haimes 2008).
However, neither in the current practice, nor in most of the available scientific contributions (Thermou and Elnashai 2006), seismic retrofitting of RC frames is approached as an optimisation problem. In fact, it is rather addressed as a merely technical issue; any considerations about optimisation (often restricted to the “economic” standpoint) are left to the engineering judgement and, hence, they are not part of a systematic analysis. This is because this optimisation problem could not be approached by means of analytical techniques commonly employed in structural engineering, but it needs meta-heuristic techniques capable of handling objective functions with no closed-form analytical expressions and possibly multi-criteria performance objectives.

Therefore, this paper outlines a general procedure based on a genetic algorithm inspired to the well-known Darwin’s “evolution of species” concept, based on the “survival of the fittest” rule (Darwin, 1859). Although some pioneering applications of these techniques are already available in the field of structural engineering, they are mainly restricted to the design of new structures (Papadrakakis and Lagaros 2002). The following sections summarise the main aspects of the proposed genetic algorithm and its application in the rational design of retrofitting interventions of existing RC frames.

OVERVIEW OF THE PROPOSED GENETIC ALGORITHM

Problem statement and formulation

As is well-known, the objective of seismic retrofitting can be conceptually described by the following Limit State (LS) function $g_{LS}$:

$$g_{LS,i} = C_{LS,i} - D_{LS,i} \geq 0$$

(1)

where $C_{LS,i}$ is the capacity of the strengthened structure at the $i$-th relevant LS and $D_{LS,i}$ the corresponding demand. Capacity and demand mentioned in Eq. (1) are generally intended in terms of either displacements (for ductile mechanisms) or forces (for brittle mechanisms). Moreover, this relationship should be checked at all the relevant LSs. As a matter of principle, Eq. (1) is not verified in seismically vulnerable.

Therefore, a combination of retrofitting techniques belonging to the so-called member- and structure-level techniques are considered for acting on the two terms of Eq. (1) (Martinelli et al. 2015). Specifically, if $x$ is the vector of design variables defining the generic combination of these techniques intended as retrofitting intervention, the optimal solution $\bar{x}$ may be determined by solving the constrained optimisation problem conceptually defined as follows:

$$\bar{x} = \arg \min_{x} \left[ f(x) \right]$$

$$g_{LS,i}(x) \geq 0 \quad \forall i = 1...n_{LS}$$

(2)

where $f(x)$ is the selected objective function and $L_{S,i}$ is the $i$-th LS out of the $n_{LS}$ of relevance.

As already mentioned, assuming the total direct cost of intervention is the simplest choice for the objective function. This is defined by considering the cost of both member- and structure-level techniques as conceptually denoted below:

$$f(x) = \left[ C_{loc}(x) + C_{glob}(x) \right] \cdot \Phi \left( \max_{i} \left| g_{LS,i}(x) \right| \right)$$

(3)

where $\Phi(\bullet)$ is a penalty function that modifies the nominal cost function of local and global interventions described by the vector $x$ (referred to as $C_{loc}(x)$ and $C_{glob}(x)$, respectively) if the inequality in Eq. (2) in not verified. It is worth highlighting that cost functions should take into account both demolition and reconstruction operations requested by both member- and structure-level techniques. Furthermore, the cost of structure-level intervention should include the possible strengthening of the original foundations, which often require the realisation of micropiles.

Representation of “individuals”

Figure 1 proposes a conceptual map of the numerical procedure proposed to solve the constrained optimisation problem described by Eq. (2). It starts with generating randomly an initial population of $N_{ini}$ individuals: a simple chromosome-like array of bits represents the single “individual” $x$ of that population.
Binary coding is adopted in the current implementation of the algorithm (Biondini, 1999). Since \( x \) includes the description of both member- and structure-level techniques, it is built by concatenating single variables that describe both types of interventions. On the one hand, the first part of the binary code representing the generic individual \( x \) includes the number of FRP layers confining each column of the existing frame. On the other hand, the second part of the \( x \) code represents the profiles adopted for realizing the first level of the concentric steel bracings adopted as a structure-level technique: the sections of the upper level profiles are linked to the former by means of a consistent design criterion chosen among those available in the literature. This leads to have each bracing described by only one steel profile, which is quite convenient in order to keep the cardinality of \( x \) as small as possible.

Based on these assumptions, the binary coding of an individual \( x \) for a simple three-storey frame with rectangular plan takes the form expressed in Figure 2.

Figure 2 Example of binary genotype for coding a retrofitting solution

It is worthwhile to mention here that two bits are adopted for each column: since each bit can take a value either 0 to 1, only \( 2^2 = 4 \) alternative confinement solutions can be taken into account: “00” would represent the absence of confinement, whereas “01”, “10” and “11” would correspond to one, two and three FRP layers, respectively. Moreover, since the maximum number of member-level interventions is equal to the number of columns, a total number of \( 2 \times N_{\text{col}} \) bits are allocated for describing them (Figure 2). The modified Kent and Park model available in OpenSEES (Mazzoni et al. 2006) is taken into account for simulating the effect of FRP confinement on concrete sections (Figure 3): it includes both strength and ductility enhancement due to the confinement.

Figure 3 Model by Kent and Park (1972) for confined concrete sections.

Furthermore, since steel bracings are supposed to be only realised between each couple of columns connected by a beam, the maximum number of bracing systems is equal to the number of beams \( N_{\text{beams}} \) in the first floor. Three bits are employed for codifying structure-level techniques (Figure 2). Consequently, there are only \( 2^3 = 8 \) possible phenotype solutions, which identify the section of steel bracings at the first level. They can range from “000” would mean absence of bracing system whereas “111” would point to the 7th profile in a list of industrial steel members potentially available for designing retrofitting interventions.
Seismic Analysis of “individuals” and evolution criteria

Static NonLinear Analyses (PushOver) are employed to quantify the values of $g_{LS,i}$ through Eq. (1) for a given “individual” $x$ of the population of technically feasible interventions (Figure 4). More specifically, capacity models adopted by current seismic codes (CEN 2005) are considered for determining $C_{LS,i}$, whereas $D_{LS,i}$ is determined by means of the well-known N2-Method (Fajfar 1999): Figure 5 depicts the main operations needed for determining the value of $D_{LS,i}$ requested for quantifying $g_{LS,i}$.

The genetic algorithm evolves through three operators until the counter of population reaches a maximum fixed number. These operators aim at creating new solutions of the retrofitting problem by selection, combination or mutation of the current population.

![Figure 4 Pushover analysis and capacity curve of a sample structure](image)

![Figure 5 Construction of Capacity Spectrum of equivalent SDOF and determination of “performance point”](image)

The first genetic operator is used to select “parents” among a mating pool according to their fitness. The fitness function $F$ of each individual $x_k$ can be defined as follows:

$$F(x_k) = \min_{h=1,N_{ind}} \frac{f(x_k)}{f(x_h)} ,$$

where $N_{ind}$ is the (invariant) number of individual forming each generation (fixed to 50). The function $F(x_k)$ “measures” the performance of individuals solution in the problem domain: it tends to zero for extremely “unfit” individuals, whereas it is equal to one for the fittest individual of each generation. Each individual “compete” with the other ones within the same generation and its probability to “survive” and, hence “reproduce” its features in the successive generation is defined as a function of its fitness:

$$p(x_k) = \frac{F(x_k)}{\sum_{h=1}^{N_{ind}} F(x_h)} .$$

Therefore, the selection procedure is implemented through the so-called “modified roulette-wheel” rule (Lipowski et al 2012), whose circular sections are proportional to the probability of each individual (Figure 6).
Hence, the individual $x$ characterised by higher fitness value has a larger range in the cumulative probability values and, consequently, has a higher probability of being selected for the mating pool. With the aim of selecting $n$ strings, $n$ random numbers are generated in the $[0, 1]$ interval and, for each one of them, the individual whose segment includes the random number is actually selected.

The second operator combines segments of selected strings, delimited with red dashed lines in Figure 7, into new “offspring” solutions by exchanging their genetic information between successive crossover points (“multi-point” crossover): the influence of the number of crossover points on the resulting efficiency is a key aspect, whose discussion is outside the scope of this paper. For further details, Readers may refer to De Jong and Spears (1992).

Finally, the third operation is mutation, which “mutates” the bit from 0 to 1 or vice versa if a fixed probability test is passed. This probability, called mutation rate $p_m$, is usually fairly small and, in the present proposal, is set to 0.02. In detail, a “coin toss” mechanism is employed: the bit is mutated if a random number between zero and one is less than the aforementioned mutation rate (Table 1).

![Figure 6 Roulette wheel in the selection scheme](image)

<table>
<thead>
<tr>
<th>Individual Chromosomes</th>
<th>Probability</th>
<th>Cumulative P.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 [010101010101011011000110]</td>
<td>0.24</td>
<td>0.24</td>
</tr>
<tr>
<td>2 [110011110101111111010011]</td>
<td>0.08</td>
<td>0.32</td>
</tr>
<tr>
<td>3 [0001010001110010110111]</td>
<td>0.39</td>
<td>0.71</td>
</tr>
<tr>
<td>4 [111100000101011111000010]</td>
<td>0.10</td>
<td>0.81</td>
</tr>
<tr>
<td>5 [011100010111010111010111]</td>
<td>0.19</td>
<td>1.00</td>
</tr>
</tbody>
</table>

![Figure 7 Crossover operator applied to couples of “parent” individuals](image)

Table 1 Mutation operator applied to “offspring” string

<table>
<thead>
<tr>
<th>Old chromosome</th>
<th>1</th>
<th>0</th>
<th>1</th>
<th>0</th>
<th>1</th>
<th>1</th>
<th>0</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Random numbers</td>
<td>0.007</td>
<td>0.703</td>
<td>0.325</td>
<td>0.245</td>
<td>0.802</td>
<td>0.008</td>
<td>0.023</td>
<td>0.472</td>
</tr>
<tr>
<td>New chromosome</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

SAMPLE APPLICATION

A regular three-storey frame is considered as a case-study with the aim to show a preliminary example of the potential of the proposed algorithm: three bays are present in the longitudinal direction (represented in Figure 8), whereas there is only one bay in the transverse direction. Therefore, it is worth highlighting that since there are 8 columns and 10 beams, the total length on the binary genotype of each individual is $8 \times 2 + 10 \times 3 = 46$ bits.

![Figure 8 Sample existing structure (a) and optimal retrofitting solution found by procedure (b)](image)

The frame is assumed to have rigid connections and fixed supports. The modulus of elasticity of steel is equal to 210 GPa and the yield stress is $F_y = 350$ MPa. The uniaxial Kent-Scott-Park model with degraded linear
unloading/reloading stiffness and no tensile strength is assumed for modelling the behaviour of concrete and the effect of confinement (Mazzoni et al 2006). The permanent load is taken as $G=5.00 \text{kN/m}^2$ and the live load is taken as $Q=2.00 \text{kN/m}^2$. The gravity loads are contributed from an effective area of $75 \text{m}^2$. Nonlinear beam-column elements with spread plasticity is employed for the FE model employed to perform pushover analysis. Fiber approach is used to account for material inelasticity. Each structural element is discretized into 5 sections located at the Gauss quadrature integration points. Each section is divided into a number of fibers, which comply to beam kinematics and each follows its own constitutive model.

Figure 9 depicts the evolution of the solution search throughout generations: the objective (cost) function starts from a cost of 94127 € and decreases progressively. As expected, the curve shows a very steep slow over the first generations and a slower and slower convergence, often characterised by a staircase shape, towards the final convergence, which is supposed to have been achieved as after 150 generations, as no further improvement is observed in $f(x)$ over the last 18 iterations. In the example under consideration, the optimal solution came up to consist of a simple concentric steel bracing and no local FRP interventions were actually required (Figure 8b) in the case under consideration. Figure 10 shows the optimal genotype in the 150th population and whose phenotype has a cost of 14282 €, the lowest of all previously processed solutions.

![Figure 9 Convergence history: objective function vs generations](image)

![Figure 10 Genotype of fittest individual in the last population](image)

CONCLUSIONS

This paper has proposed a general procedure intended at optimising seismic retrofitting of existing structures by combining member- and structure-level techniques. Despite the limitations of the current implementation (which, for instance, does not consider relevant aspects, such as the cost of the possible foundation strengthening, and assumes the actual cost as a simple objective function), the proposed procedure is a promising tool for approaching a subject of current relevance whose solution is generally left to the engineering judgement. The proposed application confirms the potential of the proposed procedure, as it demonstrates that the implemented genetic algorithm is capable of finding a solution characterised by a cost significantly lower than the initial trial solution. However, the complete implementation of this procedure is still under development: the work ahead should not only be intended at including the aforementioned aspects that are not currently accounted for, but also at enhancing the computational efficiency of the programming code.

REFERENCES


HYSTERIC BEHAVIOR OF CONCRETE COLUMNS REINFORCED BY STEEL-FRP COMPOSITE BARS UNDER QUASI-STATIC LOADING

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ABSTRACT

Concrete columns reinforced by steel-fiber reinforced polymer (FRP) composite bars (SFCB) may have a stable post-yield stiffness and good anti-corrosion performance. The hysteric behavior of three reinforced concrete (RC) columns under quasi-static loading was examined with an axial compression ratio of 0.11, and the longitudinal reinforcement types included ordinary steel bar and steel-basalt FRP (BFRP) composite bar (SBFCB) with different post-yield stiffnesses. The test results showed that the post-yield stiffness of the ordinary RC column was negative, whereas those of the other two SFCB columns were positive. After the columns yielded, the plastic strain of the ordinary RC column was concentrated at the column base while that of the SFCB columns were spread over a longer region, which may result in a smaller residual strain. The ductility of the SFCB columns was even larger than that of the RC column, which demonstrated that the elongation rate of BFRP could satisfy the lateral drift demand of a concrete column under an earthquake excitation.

KEYWORDS

Concrete column, steel-FRP composite bar, hysteric behavior, strain development.

INTRODUCTION

The Economic Development Research Group (2011) demonstrated that the deterioration of steel in reinforced concrete (RC) bridges and in other facilities of the American transport infrastructure cost approximately $139 billion in 2010. Fiber reinforced polymer (FRP) is a type of composite with a better anti-corrosion performance in concrete structures (Bakis et al., 2002). Mufti et al. (2007) reported that specimens reinforced by glass FRP (GFRP) bars in Canadian concrete bridges or in marine structures maintained an acceptable performance for 5~8 years. Because the bonding between the GFRP and the concrete was nearly perfect, their results prompted the Canadian Highway Bridge Design Code (CAN/CSA-S6-00 (R2005)) to use GFRP bars as one of the primary means of reinforcement in concrete structures.

Due to the elasticity and brittleness of an FRP bar, the stress level when using a GFRP bar as the prestressing tendon is limited to 25% of its ultimate strength (CAN/CSA-S6-00 (R2005)). Because of the low elastic modulus of GFRP or the high price of carbon FRP (CFRP), the application of FRP bars to concrete structures is still not widely accepted by engineers. By combing steel and FRP, a better performance and a more acceptable cost can be achieved. Aiello et al. (2002) used an FRP bar to replace a corner steel bar in concrete beams with an equal area, and the results showed that the steel bar could reduce the crack space and decrease the crack width. Lau and Pan (2010) conducted a test of 12 concrete beams, in which the reinforcement was from a steel bar, a GRP bar, or a steel/GFRP bar. It was found that the minimum reinforcement ratio for a beam reinforced by FRP could be reduced by 25% according to ACI 440.1R-06 (ACI 2006). Pang et al. (2015) focused on the calculation method for determining the load-displacement relationship of the hybrid reinforced concrete beam, and a new ductility coefficient was proposed for the evaluation of the deformation capacity. Kara et al. (2015) analyzed the test results of 46 beam specimens, which included beams reinforced by hybrid steel-FRP bars and concrete beams strengthened by near-surface mounted FRP bars. The failure modes of the concrete beams were investigated based on the ratio of the FRP reinforcement to the total reinforcement.
Concrete structures reinforced by hybrid steel-FRP bars still have a great risk of deterioration failure, a better anti-corrosion performance can be obtained by steel-FRP hybrid bars, which are a new type of rebar with an inner steel bar and an outer FRP. Sixteen hybrid reinforced concrete beams were tested by Nanni et al. (1994), and their results showed that the load-displacement curves of the hybrid reinforced beams can be calculated using the traditional RC theory, and the hybrid beams developed fewer cracks and had a larger crack width. Saikia et al. (2005) tested concrete beams with longitudinal steel-GFRP composite bars and GFRP stirrups, and a slip between the inner steel bar and the outer GFRP happened due to underdeveloped manufacturing technology. By modifying a pultrusion machine for FRP bars, a new type of steel-FRP composite bar (SFCB) can be obtained with good quality (Wu et al., 2010), and the ratio of FRP to steel can be specified by the customer (Sun et al., 2012). An experimental study on a concrete column reinforced with steel-FRP composite bars under quasi-static loading was conducted, and the hysteric behavior and the strain development of an ordinary RC column and SFCB columns are analyzed in this paper.

EXPERIMENTAL PROGRAM

Specimen Design and Test Setup

The longitudinal reinforcement ratio $\rho$ of an RC column is defined by the total area of the rebar over the gross section area. For a concrete column reinforced by a hybrid composite bar, an equivalent longitudinal reinforcement ratio ($\rho_{sf}$) is proposed with regards to a steel reinforced concrete column (Equation (1)), and the post-yield stiffness ratio of a SFCB ($r_{sf}$) can be defined by Equation (2) (Sun et al., 2014),

$$\rho_{sf} = \frac{E_f A_f + E_s A_s}{E_f A_{fg}}$$

(1)

$$r_{sf} = \frac{E_f A_f}{(E_f A_f + E_s A_s)}$$

(2)

where $E_f$ and $A_f$ are the elastic modulus and the total cross-section area of SFCB’s outer FRP, respectively; $E_s$ and $A_s$ are the elastic modulus and the cross-section area of the inner steel bar, respectively; and $A_g$ is the gross cross-section area of the concrete column. As a result, columns with the same $\rho_{sf}$ will have the same initial stiffness.

Three concrete columns were designed, and the corresponding specimen numbers and the mechanical properties of the reinforcements are presented in Table 1. The notation ‘C-S12’ denotes that the longitudinal reinforcement of the column is a steel bar with a 12 mm diameter. The notation ‘C-S10B49’ denotes that the concrete column is reinforced by a type of SFCB made from a 10-mm diameter inner steel bar longitudinally compounded with 49 bundles of 2400 tex basalt fibers; ‘tex’ is the weight (g) of one fiber bundle per kilometer. The notation is similar for C-S10B85, which has 85 bundles instead of 49.

<table>
<thead>
<tr>
<th>Column number</th>
<th>Reinforcement Diameter (mm)</th>
<th>Elasticity modulus (GPa)</th>
<th>Post-yield stiffness ratio</th>
<th>Yield strength (MPa)</th>
<th>Ultimate strength (MPa)</th>
<th>$\rho_{sf}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-S12</td>
<td>12.00</td>
<td>200</td>
<td>0.189</td>
<td>400</td>
<td>529.60</td>
<td>1.09%</td>
</tr>
<tr>
<td>C-S10B49</td>
<td>16.16</td>
<td>111.3</td>
<td>0.189</td>
<td>208.2</td>
<td>691.42</td>
<td>0.96%</td>
</tr>
<tr>
<td>C-S10B85</td>
<td>18.00</td>
<td>94.6</td>
<td>0.266</td>
<td>189.2</td>
<td>544.08</td>
<td>1.09%</td>
</tr>
</tbody>
</table>

Table 1 Specimen numbers and mechanical properties of the longitudinal reinforcements

![Figure 1 Test set up and geometry of concrete column](image-url)
The detailed dimensions of the columns and the loading patterns are shown in Figure 1. The shear span ratio was five, and an electrohydraulic servo test system applied a controlled vertical load of 200 kN for this test. To minimize the horizontal friction forces caused by the uniaxial compression at the column cap, tetrafluoroethylene plates and a pulley were positioned between the vertical loading actuator and the reaction frame. The friction coefficient between the tetrafluoroethylene plates and the reaction frame was approximately 0.03, and the friction coefficient between the pulley and the tetrafluoroethylene plates was less than 0.03. The compressive strength of the 150×150×150 mm concrete cubes over 28 days was 36.64 MPa, and the corresponding cylinder compressive strength was 29.31 MPa. Therefore, the axial compression ratio was 0.11.

**Loading Procedure and Measurements**

The horizontal cyclic loading on the column cap was controlled by a lateral force prior to column yielding, with a loading gradient of 10 kN for each step. After yielding, the loading was controlled by displacement (7 mm in this experiment), with each displacement cycled three times. The test measurements included the column cap force versus lateral displacement curves and the longitudinal reinforcement’s strain distribution, which was measured by seven strain gauges along the longitudinal bar.

**HYSTERETIC BEHAVIORS**

The load-lateral displacement curves and the failure modes of the columns are shown in Figure 2. Flexural failure occurred in all specimens. During loading, the concrete cover near the column base initially cracked, followed by the yielding of the longitudinal steel bars and the inner steel bars of the SFCBs. Spalling of the concrete cover in the column base subsequently occurred with a rupture or a partial rupture of the FRP. The characteristic values of the concrete columns are listed in Table 2, where \( V_C, V_y, \) and \( V_p \) are the crack load, yield load, and ultimate load, respectively; \( \delta_u, \delta_y, \) and \( \delta_p \) are the corresponding lateral displacements; and \( \mu \) is the displacement ductility coefficient \((\delta_u/\delta_y)\). The yield loads and displacements were determined by the first yielding of the longitudinal reinforcement. The ultimate load displacement was the point at which the load capacity decreased to 85% of the peak load. The post-yield stiffness ratios of the columns \((RC_u)\) are presented in Table 2 and are defined by Equation (3):

\[
RC_u = \frac{K_2}{K_1}
\]

where \( K_1 = V_y/\delta_y \) and \( K_2 = (V_u - V_y)/\delta_u - \delta_y \) are the initial and post-yield stiffnesses of the concrete column. To compare the \( RC_u \) of different columns at similar drift ratios, the \( RC_u \) of the hybrid reinforced columns is calculated at the same ductility level \((\mu = 6.28)\).

**Table 2 Characteristic values of the concrete columns**

<table>
<thead>
<tr>
<th>Column number</th>
<th>( \delta_u ) (mm)</th>
<th>( V_C ) (kN)</th>
<th>( \delta_y ) (mm)</th>
<th>( V_y ) (kN)</th>
<th>( \delta_p ) (mm)</th>
<th>( V_p ) (kN)</th>
<th>( RC_u )</th>
<th>( \mu ) ( \delta_u/\delta_y )</th>
</tr>
</thead>
<tbody>
<tr>
<td>S12</td>
<td>6.70</td>
<td>70</td>
<td>10.25</td>
<td>78.97</td>
<td>42.10</td>
<td>67.12</td>
<td>-0.78</td>
<td>6.28</td>
</tr>
<tr>
<td>C-S10B49</td>
<td>6.57</td>
<td>67</td>
<td>35.36</td>
<td>79.14</td>
<td>46.70</td>
<td>67.27</td>
<td>2.85</td>
<td>7.11</td>
</tr>
<tr>
<td>C-S10B85</td>
<td>7.00</td>
<td>73</td>
<td>55.09</td>
<td>88.70</td>
<td>80.90</td>
<td>75.40</td>
<td>3.68</td>
<td>11.56</td>
</tr>
</tbody>
</table>

The first flexural crack of C-S12 was observed at approximately 150 mm from the column foundation when the horizontal load was approximately 30 kN; the corresponding crack width was approximately 0.06 mm. The load of the turning point in the hysteretic curve (Figure 2(a)) caused by the cracks was approximately 50 kN. Six corresponding cracks were observed near the column base, with a maximum crack width of 0.08 mm. The average lateral load from pushing and pulling was 72.5 kN when the lateral displacement at the column cap reached 14 mm; the subsequent lateral load began to decrease with an increase to the lateral displacement. When the displacement reached 42 mm, the load capacity decreased to 85% of the peak load, i.e., the failure point. When the lateral displacement at the cap increased to 49 mm, a partial spalling of the concrete at the column base was observed in the second loading loop. As the loading continued, the buckling of three longitudinal reinforcements at the A-side and one at the B-side occurred when the lateral displacement was 56 mm (Figure 2(b)).

The first flexural crack of C-S10B49 occurred at approximately 250 mm from the column foundation when the lateral load was approximately 40 kN; the crack width was approximately 0.02 mm. The load at the turning point in the hysteretic curve (Figure 2(c)) caused by cracking was approximately 54 kN. Fifteen cracks appeared on the two sides of C-S10B49, of which the maximum crack was observed at 150 mm from the column foundation with a width of 1 mm. The outer FRP of the SFCB at the corner of the A-side began to rupture gradually after the lateral displacement reached 33 mm (Figure 2(d)), with the maintained lateral load decreasing with an increasing lateral displacement.

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As for C-S10B85, the turning point of the hysteretic curve caused by cracking was approximately 56 kN (Figure 2(e)), and three cracks were observed on the column base after this loading step. The maximum crack was located at a 50-mm height from the column foundation with a 0.15-mm crack width. When the lateral displacement of C-S10B85 reached 21 mm, a slight spalling of the concrete cover at the column base occurred. The concrete cover in the four corners of the column base completely crumbled when the lateral displacement reached 42 mm. After the column yielded, the load-displacement of C-S10B85 remained stable until the lateral displacement reached 69 mm (Figure 2(f)). This indicates a good ductility and deformation ability.

STRAIN DEVELOPMENT OF THE LONGITUDINAL REINFORCEMENT

Strain gauge location

Figure 3 shows the strain distribution and strain gauge locations for the longitudinal reinforcement of an ordinary RC column and for an SFCB reinforced column. Strain gauges Nos. 1–3 were in the anchorage region, and Nos. 3–7 were all above the interface of the column foundation and the column base. The curvature of the column section can be obtained from the tensile and compressive strains, and the corresponding column lateral
displacement can be integrated by the curvature along the column height. The lateral drift of a concrete column includes flexural deformation, shear deformation, and slip. For the columns in this paper, the flexural deformation is the dominant component of the column cap displacement.

(a) Stain distribution on longitudinal bar  
(b) Strain gauge location

Figure 3 Strain distribution and location of the seven strain gauges

Peak compressive strain was smaller than 1300 \( \mu \varepsilon \)

(a) Strain-lateral displacement (B2)  
(b) Strain-lateral force (B2)

(c) Strain-lateral displacement (B4)  
(d) Strain-lateral force (B4)

(e) Strain-lateral displacement (B7)  
(f) Strain-lateral force (B7)

Figure 4 Strain development of steel bar in C-S12
Strain development

The strain-lateral displacement curves and the strain-lateral force curves of C-S12 at strain gauges B2/B4/B7 are shown in Figure 4. The strain of B2 is kept ‘constant’ (approximately 2550 $\mu$e) after the lateral displacement reached 10 mm, the peak compressive strain decreased with an increase in the lateral displacement, and the compressive strain of B2 was smaller than 1300 $\mu$e during the reversed loading (Figure 4(a)). Before the column cracked, the strain-lateral force curve of B2 was almost linear before the load reached 50 kN (Figure 4(b)), the corresponding lateral displacement was 2.42 mm and the strain of B2 was 345 $\mu$e. After cracking, the strain of B2 increased dramatically with an increase in the lateral load. When the steel bar reached 2000 $\mu$e, the lateral displacement was approximately 6.7 mm. The strain of B4 was approximately 3200 $\mu$e when C-S12 yielded at the member level (Figure 4(c)), and a similar turning point can be found in the hysteric curve of C-S12. The general strain-lateral force relationship of B4 (Figure 4(d)) was similar to that of B2, and the peak strain of B4 was kept constant (approximately 3200 $\mu$e) while the lateral displacement was increasing. Because strain gauge B7 was in the elastic region, the strain kept increasing with an increase in lateral displacement (Figure 4(e)), which indicated that the section moment was increasing. However, Figure 4(f) illustrated that the lateral force was decreasing with an increase in the section moment, and as a result, the incremental moment was partially caused by the P-Δ effect.

Because the strain-lateral displacement/load curves of C-S10B49 and C-S10B85 were similar, Figure 5 only presents the strain development trend of C-S10B49. The B3 strain for C-S10B49 had an almost linear relationship to the lateral displacement (Figure 5(a)), even after the yielding of C-S10B49, whereas the B4 strain for C-S12 presented a ‘yield plateau’ after column yielding. Both strain-lateral force curves of B2 and B3 of C-S10B85 are illustrated in Figure 5(b), the tensile strain of B3 was much larger than that of B2, whereas the compressive strain curves of B2 and B3 were similar to each other. Figure 5(c) and Figure 5(d) present the development trend of the B7 strain for C-S10B49 and C-S12. Because B7 was in the column’s elastic region, the strain was proportional to the section force.

![Figure 5 Strain development of steel bar in C-S10B49](image)

Form the strain-lateral displacement curves of C-S12 (B4) and C-S10B49 (B3), as shown in Figure 5(a), the residual displacement of C-S12 increased with the loading process, whereas the residual strain (B3) and the residual displacement of C-10B49 was close to zero, even when the peak strain of C-S10B49 (B3) reached 7000 $\mu$e.
CONCLUSIONS

Three concrete columns under cyclic loading were tested in this paper. The main parameter was the post-yield stiffness of the longitudinal reinforcement, and the axial compression ratio was 0.11. Conclusions are as follows: (1) the post-yield stiffness of the ordinary RC column was negative, whereas that of the SFCB columns were positive, and the post-yield stiffness can be designed to optimize the FRP/steel ratio in SFCB; (2) the ductility of the SFCB columns was greater than that of the RC column because the load carrying capacity of C-S12 decreased faster for the RC column than the SFCB columns; and (3) the plastic strain of C-S12 was concentrated at the column base, whereas that of the SFCB columns could spread the plastic strain over a longer region and decrease the residual strain after loading, and the corresponding seismic resilience of the structures can be enhanced.

ACKNOWLEDGMENTS

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SEISMIC BEHAVIOR OF GLASS FRP WALL PANELS

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ABSTRACT

Glass FRP (GFRP) panels have been used for structural applications due to their advantages of lightweight, corrosion resistance and construction-easiness. This paper evaluates their seismic performances as wall panels, based on combined shaking table tests and finite element (FE) modeling. A scaled GFRP wall panel was tested on a shaking table subjected to harmonic base motions with frequencies ranging from 10 to 15 Hz. A mass that was twice of the panel’s self-weight was attached to the top of the panel to simulate its supported seismic weight. The panel remained undamaged under a peak base acceleration of 3.5g. Numerical simulation was conducted in Abaqus using Rayleigh damping for comparison with the test results. Good correlations in natural frequency and other dynamic responses were achieved. This research shows good potential of using GFRP panels as seismic-resistant structural members.

KEYWORDS:
FRP, wall panel, seismic behaviour, shaking table test, finite element modeling.

INTRODUCTION

Fiber reinforced polymer (FRP) materials have been widely used in civil engineering field. While they are more commonly used to strengthen existing structures, all FRP structures have been increasingly used in recent years since they are easy to retrofit, can achieve time and labor savings and reduce the self-weight, which allows more design flexibility. While early research work mainly focused on static behavior of GFRP structures (e.g. Clarke, 1996; Davalos et al. 1996), their dynamic behavior has been studied recently mainly through experimental investigations (Boscato and Russo, 2009, 2011, 2012, 2014, 2016), including large-scale space frames (Boscato et al. 2015; Xiao et al. 2016), bridge deck panels (Burgueño et al. 2001; Wight et al. 2006; Alampalli, 2006; Reising et al. 2001), beams (Bank, 1989), etc. However, there is still no study available on the seismic behavior of GFRP panels as load-bearing walls, which is the objective of this study.

In this paper, a GFRP wall panel was tested under harmonic ground motions using a shaking table. The displacements and dynamic characteristics of walls were then correlated with the results from Finite Element (FE) analysis. Both results show that GFRP panels have a potential to be used as seismic-resistant structural members.

Figure 1 Geometry profile of GFRP panel wall
PANEL GEOMETRY AND MATERIAL PROPERTIES

The panel used in this study was a Composolite® building panel provided by Strongwell®. It was 61 cm wide and 122 cm long and made of glass fiber using a pultrusion process. The geometry of the panel is shown in Figure 1. As provided by the manufacturer, the out-of-plane and in-plane moment of inertia were $6.62 \times 10^4$ cm$^4$ and $176 \times 10^4$ cm$^4$, respectively. The thickness of the GFRP panel was 0.297 cm for the outer wall, and 0.218 cm for the separation between the cells.

MATERIAL PROPERTIES

A bending test was conducted to determine the stiffness of the panel. Strain gauges were installed at the mid-span of the simply-supported panel (Figure 2). When subjected to a concentrated load of 454 N at the mid-span, the maximum strain at the tension side of the panel was 83 με. This result correlates well with FE result, where the maximum strain is 87 με. The out-of-plane stiffness calculated from the test result is $1.17 \times 10^9$ N cm$^2$. Based on the deflection reported by the manufacturer for a beam with a 183 cm span, the stiffness $EI$ can be calculated to be $1.23 \times 10^9$ N cm$^2$. Therefore, the material properties provided by the manufacturer can be used in the following analysis, where the elastic modulus and Poisson’s ratio are 18.6 GPa and 0.27, respectively.

A free vibration test was also done to obtain the natural frequencies and damping ratio of the GFRP panel. The bottom of the wall panel was fixed to the ground, and a plastic hammer was used to excite the panel at the top, as shown in Figure 3. An accelerometer and an LVDT were installed at the top of the panel to record its acceleration.
and displacement, as shown in Figure 4a. Using Fast Fourier Transform (FFT) to analyze the displacement data, the first three natural frequencies can be calculated as 12.7 Hz, 31.2 Hz and 39.1 Hz, respectively, as shown in Figure 4b. The damping ratio of the structure can also be determined by computing the percentage of amplitude reduction after a certain number of cycles, which is 0.16. It is relatively large compared to typical damping ratios for reinforced concrete (0.05) and steel structures (0.02).

SHAKING TABLE TEST

The shaking table test setup is shown in Figure 5. The GFRP panel was connected to the shaking table using bolts and angles, which can be treated as a rigid connection. A steel block of 24.5 kg was connected to steel angles, which was firmly bolted to the edges at the top of the panel. The attached mass was to simulate the seismic weight of a flat roof in a typical low-rise building, which includes the total dead load and 20% of snow load. Both tests with and without attached mass were conducted to evaluate the dynamic responses of the wall panel.

The GFRP panel was subjected to harmonic ground motions of 10.1 Hz and 15.1 Hz. Key parameters of the harmonic ground motions in the experiment are described in Table 1. Three accelerometers and LVDTs were installed at the bottom, middle and top of the panel, respectively, in order to measure the panel’s displacements and accelerations in the vibration direction. Since the shaking table is displacement-controlled, the maximum displacements of ground motions under different frequencies were about the same, while the maximum accelerations measured during the tests differed. As a result, although the ground motion displacement and displacement at different locations of the panel was harmonic, as shown in Figure 7, the ground acceleration was not harmonic, as shown in Figure 6. Instead, the maximum acceleration is greater than the average value of peak accelerations for all cycles, as can be seen from Figure 6.

<table>
<thead>
<tr>
<th>Ground motion</th>
<th>Frequency</th>
<th>Maximum displacement</th>
<th>Maximum acceleration</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10.1Hz</td>
<td>1.83 mm</td>
<td>1.4 g</td>
</tr>
<tr>
<td>2</td>
<td>15.1Hz</td>
<td>1.94 mm</td>
<td>3.5 g</td>
</tr>
</tbody>
</table>

Figure 5 Shaking table test setup

Figure 6 Acceleration of 15.1 Hz ground motion

a) 10.1 Hz ground motion b) 15.1 Hz ground motion.
TEST RESULT AND ANALYSIS

Figure 7 shows the displacements at the top of the wall panel when subjected to different ground motions provided by the shaking table. Since the shaking table movements are nearly the same under all frequencies as explained above, the deformation of the wall panel can be represented by the displacement at the top of the wall. Since the hollow sectioned GFRP panel is light, the deformation of the wall panel can be represented by the displacement at the top of the wall.

Since the hollow sectioned GFRP panel is light, the displacement at the top of the wall panel is close to that from the ground motion when only the wall panel was tested. In contrast, attaching seismic weight to the wall panel can significantly increase the top displacement, which is about twice of the displacement of the wall panel without the added mass, as shown in Figure 7(b). The increase of the displacement under 15.1 Hz ground motion is much greater than that under 10.1 Hz ground motion, because 15.1 Hz ground motion provides considerably larger acceleration and is closer to the natural frequency of the wall panel.

The acceleration in the 15.1 Hz ground motion test reached 3.5g. The recorded displacements of the GFRP panel remained unchanged during the whole process, indicating that no damage occurred despite the large shear force induced by the attached seismic weight. The maximum story drift of the GFRP panel was 0.33%, which is smaller than that of steel or concrete structures under the same load, mainly because of the high stiffness provided by the panel’s internal structure and the GFRP material’s ability to remain linear under high strain.

FINITE ELEMENT ANALYSIS

FE models are constructed using Abaqus (v6.14). Shell element S4R is used to simulate the GFRP panel (Figure 8). Boscato and Russo (2009) concluded that, in vibration analysis, GFRP pultruded structural members can be treated as isotropic materials using currently available theories and computational methods. According to the bending test results, the GFRP material is linear elastic with an elastic modulus and Poisson’s ratio of 18.6 GPa and 0.27, respectively. In the FE model, the bottom of the GFRP panel is fixed except the in-plane direction, where in-plane movements are exactly the same as the ground motions used in tests. From the free vibration test, it was found that the damping ratio of the GFRP panel is about 16%, and the first and second natural frequencies are 12.7 Hz and 31.2 Hz, respectively. Linear frequency analysis in the FE model agrees well with the free vibration test, as shown in Table 2.

Rayleigh damping is included in the material properties. The damping ratio for the nth mode can be expressed as:

\[
\zeta_n = \frac{\alpha}{2\omega_n} + \frac{\beta}{2\omega_n}
\]  

Figure 7 Displacement at the top of the wall panel under different frequencies

Figure 8 Mesh of finite element model (with and without mass)
where $\xi_n$ and $\omega_n$ are the damping ratio and natural frequency for the $n^{th}$ mode, $\alpha$ and $\beta$ are Rayleigh damping parameters, respectively. If both the first and second modes are assumed to have the same damping ratio, the coefficients can be calculated from Eq. 2, with the results shown in Table 3.

$$\alpha = \zeta \frac{\omega_1 \omega_2}{\omega_1 + \omega_2}, \quad \beta = \zeta \frac{2}{\omega_1 + \omega_2}$$ (2)

<table>
<thead>
<tr>
<th>Natural frequencies</th>
<th>$\omega_1$ (rad/s)</th>
<th>$\omega_2$ (rad/s)</th>
<th>$\omega_3$ (rad/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Free vibration test</td>
<td>79.7</td>
<td>196</td>
<td>246</td>
</tr>
<tr>
<td>FE model</td>
<td>88.0</td>
<td>220.9</td>
<td>284</td>
</tr>
</tbody>
</table>

Table 2 Comparison of natural frequencies obtained from free vibration test and FE model

| Natural frequencies and Rayleigh damping parameters used in FE model |
|--------------------------|-----------------|-----------------|
| damping ratio            | $\alpha$       | $\beta$        |
| 0.16                    | 9.09           | 0.00116         |

The shaking table tests are also modeled using FE method. The errors between test and FE results are within 13% and 12% for the cases with and without attached mass (Figure 9). The error is probably because the out-of-plane displacements could occur in the shaking table tests, which is not considered in the FE model. The FE result also validates that the GFRP panel was intact during the testing.

![Figure 9. Dry wall 10.1 Hz FE simulation results](image)

**CONCLUSIONS**

Based on this study, the following conclusions can be drawn:

1. The GFRP panel in the shaking table tests exhibited good resistance to seismic load due to its high strength and stiffness despite the high intensity of input ground motion, indicating that GFRP panels have a potential to be used as seismic-resistant structural walls in low-rise buildings.

2. FE model can successfully predict natural frequencies and displacements of the GFRP panel in the tests using Rayleigh damping and assuming that GFRP material is linear elastic.

Based on the promising results from this study, further research will be done to study the performance of GFRP wall panels when they are connected to other structural members in frame structures.

**ACKNOWLEDGMENTS**

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REFERENCE

EXPERIMENTAL STUDY ON SEISMIC PERFORMANCE OF CFRP STRENGTHENED FULL-SCALE RECTANGULAR RC COLUMNS

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ABSTRACT

To investigate the seismic performance of rectangular reinforced concrete (RC) columns strengthened with carbon fiber reinforced polymer (CFRP), four full-scale rectangular RC columns were tested under combined constant axial load and unidirectional lateral horizontal cyclic load. The constant axial compression ratio for all the columns was 0.4, while the lateral cyclic displacement was applied in strong axis (i.e. 0 degree) and weak axis (i.e. 90 degree) of specimens, respectively. All the specimens have the identical section of 300 mm × 450 mm and cantilever height of 1400 mm. The failure modes, lateral load-displacement responses, ductility were compared and investigated. The test results indicated that the seismic performance of rectangular RC columns could be significantly improved after being strengthened with CFRP, especially in the strong axis direction. The failure mode of control specimen in strong axis was a more brittle shear model, while the strengthened columns exhibited a more ductile flexural mode. The ductility factor of rectangular RC columns was significantly improved in both strong and weak axis directions after being strengthened with CFRP.

KEYWORDS

FRP, rectangular RC columns, strengthening, seismic performance, full-scale test.

INTRODUCTION

The reinforced concrete (RC) rectangular columns are commonly existed in the RC frame structures. As a critical member, the severe damage of the columns under earthquake may lead to the collapse of the whole structure. Also with the modifying of the seismic design code of buildings, the seismic performance of existing RC columns is no longer satisfied the requirements of the current seismic design code. Consequently, there is an urgent needed to improve or enhance the seismic performance of these existing “substandard” RC columns. Externally bonding fiber reinforced polymer (FRP) has been widely used in the seismic strengthening or retrofitting of RC structures in the past two decades. As a result, many studies have been conducted on the seismic performance of FRP strengthened RC columns (e.g. Seible et al. 1997; Iacobucci et al. 2003; Memon and Sheikh 2005; Wu et al. 2008; Ozcan et al. 2008; Realfonzo and Napoli 2009; Gu et al. 2010; Dai et al. 2012; Ma and Li 2014; Cai et al. 2016). However, most of these studies are focus on FRP strengthened small-scale circular or square RC columns. Relatively limited experimental investigations have been conducted on the seismic performance of FRP strengthened rectangular RC columns (e.g. Ilki et al. 2009; Ozcan et al. 2010; Realfonzo and Napoli 2012; Li et al. 2013). All of these limited studies were focus on the principal strong loading direction. The influence of loading direction was not investigated. However, real structures may subject earthquake excitation in any direction. In addition, the limited studies on the RC columns under biaxial or multi-direction lateral loading indicated the loading path has significant on the seismic performance of RC columns, especially for rectangular columns (e.g. Pham and Li 2013; Rodrigues et al. 2013). To the best of the authors’ knowledge, only two experimental studies on FRP strengthened rectangular RC columns have been conducted considering the influence of lateral loading direction (e.g. Dong et al. 2013; Rodrigues et al. 2015). Against this background, this paper presents the experimental study on seismic performance of 4 full-scale rectangular RC columns with and without strengthening by CFRP. The specimens were tested under combined constant axial load and reversed cyclic lateral load in strong and weak direction, respectively. The seismic performance of columns before and after being strengthened was investigated and evaluated by comparing the failure modes, lateral load-displacement hysteretic response and ductility capacity in the strong and weak direction, respectively.
EXPERIMENTAL PROGRAM

Test specimens and materials

A total of 4 full-scale rectangular RC cantilever columns were fabricated and tested under constant axial load and reversed lateral cyclic load. The columns were designed according to the old national seismic design code of China (GBJ11-89), representing the existing substandard RC frame columns. All specimens have the same cantilever height of 1400 mm (from the top surface of footing to central of lateral loading point) and the rectangular cross section of 300 × 450 mm. Eight 20 mm diameter hot-rolled deformed steel bars were adopted for the longitudinal reinforcement which resulted in the longitudinal reinforcement ratio of 1.86%. The plain steel bars with 8 mm diameter were used as hoop steel reinforcement. According to the old seismic design code (GBJ11-89), the potential plastic hinge zones of columns should be strengthened with dense hoops reinforcement. In this study, the hoops dense area was the bottom 500 mm height of the columns. The centre to centre spacing of hoops at the dense and normal area was 100 mm and 200 mm, which resulted in the volumetric hoop reinforcement ratio of 0.6% and 0.3%, respectively. The concrete cover thickness, measured to the outside of the hoops, was 30 mm. The columns were casted in in strong concrete stub footing with the dimension of 1500 × 650× 600 mm to simulate the fixed-end boundary. The dimension and reinforcement details of the specimens are shown in Fig. 1.

![Figure 1 Specimen dimension and reinforcement details](image1)

![Figure 2 Test setup and instrumentation](image2)

All specimens were casted in one batch of ready mixed concrete. The average compressive strength of standard cylinders at the age of 28-day and testing time was 29.1 MPa and 38.2 MPa, respectively. After 28-day standard curing time, two specimens were strengthened with three layers of unidirectional carbon FRP (CFRP) sheet at the potential plastic hinge region. The height of strengthened region was 500 mm which was the same as that of hoop dense area. The corner radius was formed by securing iron sheets at the mode with the constant value of 0.1h (i.e. 45 mm) for two strengthened columns. The measured average tensile strength and elastic modulus of steel reinforcement and CFRP material is shown in Table. 1.

<table>
<thead>
<tr>
<th>Material</th>
<th>Diameter/Thickness (mm)</th>
<th>Yield strength (MPa)</th>
<th>Ultimate strength (MPa)</th>
<th>Elastic modulus (MPa)</th>
</tr>
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<tr>
<td>Steel bars</td>
<td>8</td>
<td>315.0</td>
<td>382.7</td>
<td>134654</td>
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<tr>
<td></td>
<td>20</td>
<td>430.7</td>
<td>656.5</td>
<td>214453</td>
</tr>
<tr>
<td>CFRP</td>
<td>0.167</td>
<td>-</td>
<td>4340</td>
<td>244000</td>
</tr>
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</table>

Table 1 Tested mechanical properties of steel reinforcement and CFRP

Test setup and instrumentation

A purpose designed and built loading apparatus was utilized in this test, in which the axial load can be ensured constant and moving with the upper lateral displacement of the specimens. The schematic diagram of test setup is shown in Fig. 2. The second-order effect (i.e. P-Δ effect) could be incorporated and simulated by using this test system. At the beginning of the test, the axial load of 2063 kN corresponding to axial compression ratio of 0.4 was first applied on top of each columns. Then the reversed lateral load was applied by an electro-hydraulic actuator under displacement control mode. Two specimens, with and without strengthened, were tested with the lateral load in the strong axis direction (i.e. 0°), while the remaining two were tested in the weak direction (i.e. 90°). The specimen identification convention utilized in this study was initiated with the letter L, which refers to lateral load.
The following number of 0 or 90 refers to the lateral load direction. The last letter C or R represents the without strengthened control specimens or strengthened (retrofitted) specimens, respectively. Considering the displacement capacity difference between control and strengthened specimens, two different displacement control lateral loading schemes were adopted. For control specimens, the displacement levels were incremented by 4 mm with two cycles per level, except the first level was repeated once only. For the strengthened specimens, the first and second displacement levels were also incremented by 4 mm with and repeated once, while the following levels were increased with the imposed increment of 8 mm and two cycles at each level. The tests were finally stopped when the lateral resistance of the columns reduced to 70% of the tested peak load capacity. The lateral displacement was measured by three linear variable displacement transducers (LVDTs). In addition, two LVDTs (i.e. LVDT 4 and 5) were installed vertically to monitor the potential rotational displacement of the footing, as shown in Fig. 2. The lateral force was measured by the load cell embedded within the actuator.

EXPERIMENTAL RESULTS AND DISCUSSIONS

Test Observations and Failure Modes

The comparison of the typical failure modes of control and strengthened specimens are shown in Fig. 3. For specimens tested in the strong direction, the unstrengthened control column of L0-C exhibited a brittle shear failure mode. The flexural cracks initially formed at the plastic region under the displacement level of 8 mm. With the increase of lateral displacement, the existing flexural cracks kept developing with increasing width and length and extending diagonally and more new cracks appeared. Spalling of concrete cover in the bottom corner area of column at the displacement level of 16 mm was noticed. The column brittle failed at the displacement level of 28 mm. A main diagonal shear crack extending the whole height of the column was finally formed. Severe concrete spalling occurred at the plastic hinge region of column, leading to the total exposure of steel reinforcement in one side. The internal hoops bent outward and longitudinal bars buckled, as shown in Fig. 3 (a).

![Figure 3. Failure modes](image)

After being strengthened with CFRP, the failure of the column (i.e. L0-R) changed to ductile mode. Micro cracks were occurred in the upper unstrengthened area at the displacement level of 8 mm, and kept developing and extending diagonally with the increase of displacement level. With the displacement increased to 24 mm, horizontal cracks on CFRP wraps were first noticed at the height of 150 mm above the footing surface. The width of such cracks increased up to 2 mm with the displacement level increased to 72 mm and the wrapped CFRP locally bent outward. Finally, the CFRP was ruptured at this region with the displacement level increased to 88
mm, as shown in Fig. 3 (b1) and (b2). In this case however, the lateral resistance of the column was not rapid reduced and still remained about 70% of the tested peak load capacity. To further observe the damage of the strengthened area, the wrapped CFRP was removed after test. It was found that the damage level of concrete was significantly confined compared with the control specimen, as shown in Fig. 3 (b3) and (b4). The longitudinal steel bar buckling at the CFRP rupture region was observed (see Fig. 3 (b4)).

The shear span ratio was about 4.67 for specimens tested in the weak direction. Therefore, the control specimen of L90-C finally exhibited flexural failure mode. The concrete surface of column mainly cracked horizontally due to flexural. The concrete spalling and crushing area was concentrated in the bottom 200 mm height of column at the ultimate displacement level of 36 mm, as shown in Fig. 3 (c). The concrete damage area was smaller compared with that of specimen L0-C, and the ductility was increased. The damage process and failure mode of strengthened specimen L90-R was similar with specimen L0-R (see Fig. 3 (d1) and (d2)), but the ductility was reduced. The lateral resistance reduced to about 60% of the peak load at the displacement level of 72 mm. However, no rupture of CFRP wraps was observed and the concrete of the strengthened area was found only slightly damaged after removing the CFRP wraps, as shown in Fig. 3 (d3) and (d4).

**Lateral Load-Displacement Hysteretic Curves**

The comparison of experimental lateral load-displacement hysteretic curves of control and strengthened specimens is shown in Fig. 4. It can be seen that the control specimens tested in strong and weak direction both exhibited poor hysteretic behaviour as observed by narrow hysteretic loops and rapid decrease in lateral load after peak load at small drift ratio. The lateral resistance capacity in strong direction was much larger than that of in weak direction, but the ductility capacity was reduced. After being strengthened with CFRP, the lateral resistance capacity has hardly improved but the ductility and energy dissipation was significantly improved, especially in strong direction (see Fig. 4(a)). The strengthened specimens exhibited much wider hysteretic loops and more gradual post-peak strength degradation response compared with control specimens in both directions. The strengthening technical significantly improved the seismic performance of the rectangular RC columns in strong direction.

![Experimental lateral load-displacement curves](image)

**Figure 4.** Experimental lateral load-displacement curves

**Envelop Curves and Performance Indexes**

The envelop curves of all the specimens obtained from the experimental lateral load-displacement hysteretic curves are shown in Fig. 5. In addition, the performance indexes, such as the strength and corresponding displacement of yield (i.e. \( P_y \) and \( \Delta_y \)), peak (i.e. \( P_p \) and \( \Delta_p \)) and ultimate (i.e. \( P_u \) and \( \Delta_u \)) point are presented in Table 2. The yield point was obtained based on the energy method (e.g. Cai et al. 2016). The ultimate point corresponding to the lateral resistance reduced to 85% of the tested peak load. It can be seen from Fig. 5 and Table 2 that the wrapping CFRP has slight influence on the lateral resistance capacity, but significant effect on the deformation capacity and the strength degradation. The control and strengthened specimens exhibited similar yield strength and displacement. This due to the externally bonded CFRP wraps has almost no effect on the initial stiffness of the strengthened columns. However, the yield displacement in strong direction was smaller than that in weak direction, while the yield strength was larger. The peak strength and displacement were both slightly increased in both directions after being strengthened with CFRP. For example, the peak displacement increased about 11.8% and 4.0% in strong and weak direction, while the increment in peak strength was only about 3.9% and 2.1%, respectively.
The specimens were tested in strong and weak axis direction, respectively. The test results indicated that the seismic performance of rectangular RC columns could be significantly improved after being strengthened with CFRP, especially in the strong direction. The control specimen exhibited brittle shear failure mode and flexure failure mode in strongly and weak direction, respectively, while the strengthened columns exhibited a more ductile failure mode in both directions. The lateral resistance capacity of the columns was slightly improved after being strengthened with CFRP, while the ductility capacity was significantly enhanced, especially in strong axis direction.

**CONCLUSIONS**

This paper has presented an experimental study on the seismic performance of full-scale rectangular RC columns with and without strengthening with CFRP wraps. The specimens were tested in strong and weak axis direction, respectively. The test results indicated that the seismic performance of rectangular RC columns could be significantly improved after being strengthened with CFRP, especially in the strong direction. The control specimen exhibited brittle shear failure mode and flexure failure mode in strongly and weak direction, respectively, while the strengthened columns exhibited a more ductile failure mode in both directions. The lateral resistance capacity of the columns was slightly improved after being strengthened with CFRP, while the ductility capacity was significantly enhanced, especially in strong axis direction.

**Table 2** Tested mechanical properties of steel reinforcement and CFRP

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Load direction</th>
<th>$\Delta_u$/mm</th>
<th>$P_u$/kN</th>
<th>$\Delta_y$/mm</th>
<th>$P_y$/kN</th>
<th>$\Delta_s$/mm</th>
<th>$\delta_u$/%</th>
<th>$\mu$=<em>$\Delta_u$/</em>$\Delta_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>L0-C</td>
<td>+</td>
<td>9.56</td>
<td>308.42</td>
<td>18.79</td>
<td>361.77</td>
<td>28.08</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>7.40</td>
<td>290.17</td>
<td>13.97</td>
<td>344.86</td>
<td>26.50</td>
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<tr>
<td></td>
<td>Ave.</td>
<td>8.48</td>
<td>299.30</td>
<td>16.38</td>
<td>353.32</td>
<td>27.29</td>
<td>1.95</td>
<td>3.22</td>
</tr>
<tr>
<td>L0-R</td>
<td>+</td>
<td>9.02</td>
<td>305.57</td>
<td>18.26</td>
<td>353.63</td>
<td>61.17</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>8.44</td>
<td>322.54</td>
<td>18.37</td>
<td>380.73</td>
<td>80.77</td>
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</tr>
<tr>
<td></td>
<td>Ave.</td>
<td>8.73</td>
<td>314.06</td>
<td>18.32</td>
<td>367.18</td>
<td>70.97</td>
<td>5.07</td>
<td>8.13</td>
</tr>
<tr>
<td>L90-C</td>
<td>+</td>
<td>12.18</td>
<td>226.09</td>
<td>17.90</td>
<td>263.80</td>
<td>29.28</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>9.18</td>
<td>204.17</td>
<td>18.20</td>
<td>229.17</td>
<td>29.90</td>
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<tr>
<td></td>
<td>Ave.</td>
<td>10.68</td>
<td>215.13</td>
<td>18.05</td>
<td>246.49</td>
<td>29.59</td>
<td>2.11</td>
<td>2.77</td>
</tr>
<tr>
<td>L90-R</td>
<td>+</td>
<td>9.74</td>
<td>222.67</td>
<td>18.91</td>
<td>258.23</td>
<td>35.61</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>8.53</td>
<td>205.62</td>
<td>18.62</td>
<td>245.29</td>
<td>35.48</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Ave.</td>
<td>9.14</td>
<td>214.15</td>
<td>18.77</td>
<td>251.76</td>
<td>35.55</td>
<td>2.54</td>
<td>3.89</td>
</tr>
</tbody>
</table>

It was easily noted that the most significant improvement index after being strengthened was the ultimate displacement, especially in strong direction. For example, after being strengthened with CFRP the ultimate displacement increased about 160% in strong direction, while the incensement was about 20% in weak direction. The ultimate drift ratio ($\delta_u$) of control specimen in the two directions was both around 2%, which just met exactly the limit of the ultimate inter-storey drift ratio (i.e. 2%) of RC frame structures specified by the national seismic design code of China (GB50119-2010). While the ultimate drift ratio of strengthened frame was about 2.54 and 1.27 times of the specified ultimate limited value in strong and weak direction, respectively.

The other important performance index is displacement ductility factor ($\mu$), which reflects directly the plastic deformation capacity of structures or members. It can be seen that the ductility factor in strong direction was larger than that of in weak direction for both control and strengthened specimens. After being strengthened with CFRP, the ductility factor was significantly increased in both directions, especially in strong direction. The above discussion again demonstrated that the externally bonded CFRP could effectively improve the seismic performance of rectangular RC columns, especially in strong direction.
ACKNOWLEDGMENTS

The authors gratefully acknowledge the financial support provided by the National Natural Science Foundation of China (Grant No. 51408153, No. 51478143, and No. 51278150), the National Key Basic Research Program of China (973 Program, Grant No. 2012CB026200) and China Postdoctoral Science Foundation (Grant No. 2014M551252 and No. 2015T80354).

REFERENCES


ANALYSIS OF POST-TENSIONED PRECAST SEGMENTAL BRIDGE PIERS REINFORCED WITH STEEL AND FRP BARS

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ABSTRACT

The rapid construction nature and outstanding self-centring ability make the post-tensioned precast segmental (PTPS) bridge pier a promising alternative to traditional cast-in-place piers. In this paper, PTPS piers reinforced with both steel and fibre reinforced polymer (FRP) bars were numerically analyzed. This type of pier is referred to as a hybrid bars reinforced PTPS bridge pier in this paper. The steel bars which continuously cross the segment joints (referred to as ED bars) were expected to improve the energy dissipation ability of the PTPS piers, while the FRP bars were expected to increase the post-yield stiffness and to decrease the residual displacement. Monotonic and cyclic analyses on hybrid bars reinforced PTPS piers were subsequently conducted. Three parameters were investigated, namely, the mechanical properties of FRP bars, the FRP bar reinforcement ratio and the proportion of FRP bars to ED bars. According to the analysis results in this study, the FRP bars are effective on improving the self-centring ability of the hybrid bars reinforced PTPS piers.

KEYWORDS

Segmental bridge piers, FRP bars, post-yield stiffness, residual displacement, fibre model.

INTRODUCTION

The post-tensioned precast segmental (PTPS) bridge pier is attractive due to its accelerated construction nature and the self-centring ability. As illustrated in Figure 1(a), this type bridge pier is constructed by assembling precast concrete segments with post-tensioned tendons. Ou et al. (Ou et al. 2007) proposed to reinforce the PTPS piers with longitudinal steel bars which continuously crossed the segment joints (referred to as ED bars). The investigations on such ED bar reinforced PTPS piers indicated that the hysteretic energy dissipation ability of the piers increased significantly with the ED bar reinforcement ratio (Bu et al. 2012; Ou et al. 2007; Wang et al. 2008). Furthermore, their researches also showed that the ED bar ratio should be no larger than 0.5% in order to maintain the proper ductility and to avoid over-deformation. In order to further improve the energy dissipation ability of the PTPS piers, steel and FRP bars were simultaneously used to reinforce the piers in this paper. The energy dissipation ability of the hybrid bars reinforced PTPS piers is in the focus of this paper. In order to evaluate the energy dissipation ability of the hybrid bars reinforced PTPS piers, monotonic and cyclic analyses were conducted.

Figure 1 Schematic diagram of PTPS piers

(a) Elevation of the PTPS pier
(b) The hybrid bars reinforced PTPS pier

Gravity load
Lateral displacement
post-tensioned tendons
segment
segment joint
ED bars
T-headed nut
FRP bar
ED bar
to maintain negligible residual displacement (Ou et al. 2007). However, such an upper limit is even smaller than the specification on the minimal steel reinforcement ratio for cast in place bridge piers according to the Guidelines for Seismic Design of Highway Bridges of China (MTPRC 2008). In spired by the concept of ED bar reinforced PTPS pier, the hybrid bars reinforced PTPS piers were proposed by the authors of this paper. As shown in Figure 1(b), both the ED bars and the fibre reinforced polymer (FRP) bars are adopted in this new type of pier specimen. The ED bars were expected to improve the energy dissipation ability of the PTPS piers, while the FRP bars were expected to increase the post-yield stiffness and to decrease the residual displacement. These longitudinal bars can be used in the potential plastic hinge region only in order to make this type of pier more cost-effective. In order to validate the concept of the hybrid bars reinforced PTPS pier, both pushover and cyclic analyses were conducted in this study.

SIMULATION OF THE HYBRID BARS REINFORCED PTPS PIERS

Finite Element Model

The proposed hybrid bars reinforced PTPS pier was modelled with OpenSees program, which is an open source, objected-oriented software framework (McKenna 1997). Each pier segment was simulated by one force-based beam-column element with 5 Gauss-Lobatto integration points uniformly distributed along the element (Neuenhofer and Filippou 1997). The zero-length element with a fibre section was adopted to model the behaviour of the segment joint (Zhao and Sritharan 2007). This element can take into consideration the bond slip of ED bars at segment joints by using the Bond_SP01 material of OpenSees. Note that this kind of bond slip is caused by strain penetration, instead of anchorage failure (Sritharan et al. 2000). According to Sun (Sun et al. 2014), the strain penetration effects of a composite bar made with FRP skin over a steel rod can also be approximately modelled with Bond_SP01 material. Therefore, this material was adopted in this study to consider the strain penetration effects of the FRP bars as well. Accurate stress-slip relationship of the FRP bar can be established by pull-out tests; however, such experimental investigation is beyond the scope of this paper and will be conducted in the future. A truss element which can incorporate the geometric nonlinearity was used to model the un-bonded post-tensioned tendons. Concrete02 material (Yassin 1994) and Steel02 (Menegotto and Pinto 1973) material were used to model the concrete and ED bars, respectively. It is worthy to note that the tensile strength of the concrete fibre in the joint section was defined as zero. FRP bars were modelled with uniaxial elastic material which can take into consideration the differences of mechanical properties between tension and compression. For simplicity, the elastic modulus and strength of FRP bars in compression were defined as 50% of those in tension in this study (Mohamed et al. 2014; Tavassoli et al. 2015).

Model Validation

Two sets of quasi-static tests available in the open literature were utilized to validate the finite element model of the hybrid bars reinforced PTPS pier. Three ED bars reinforced PTPS pier specimens, tested by Ou et al. (Ou et al. 2010), together with another three column specimens reinforced with both steel bars and FRP bars, tested by Ibrahim et al. (Ibrahim et al. 2015), were simulated with the aforesaid fibre model. The predicted hysteretic curves were compared with the experimental results. Due to space limitation, only two comparisons were shown in Figure 2. As shown in this figure, the simulated cyclic behaviours compared well with the tested results, implying that the numerical model used in this study was reasonably accurate.

Figure 2 Comparison of predicted hysteretic curves with test results
Parametric analysis was then carried out in order to investigate the seismic performance and the self-centring ability of hybrid bars reinforced PTPS piers. A prototype cantilever PTPS pier was designed. As shown in Figure 3, the height and the diameter of the prototype cantilever pier were 9.0m and 1.5m, respectively, resulting in a shear span ratio of 6. The PTPS pier consisted of 6 segments with the same height of 1.5m. The superstructure gravity load was 0.05$f'_cA_g$, in which $f'_c$ was the cylinder strength of the unconfined concrete and $A_g$ is the cross sectional area of the pier. In addition, the post-tensioning force was 0.05$f'_cA_g$ as well (Bu et al. 2012). The hoop reinforcement volumetric ratio for all the segments was approximately 1.0%. The variation in the mechanical properties of FRP bars, the reinforcement ratio of FRP bars $\rho_{FRP}$ and ED bar ratio $\rho_{ED}$ is shown in Table 1. Therefore, there were $2 \times 4 \times 4 = 32$ specimens in total analysed in this study. Note that the mechanical properties of BFRP bars and CFRP bars in this table were extracted from experimental investigations of Ibrahim et al. (Ibrahim et al. 2015) and Mohamed et al. (Mohamed et al. 2014), respectively.

![Figure 3 Dimensions of analysed PTPS piers](image)

<table>
<thead>
<tr>
<th>Mechanical properties of FRP bars</th>
<th>FRP bar ratio $\rho_{FRP}$ (%)</th>
<th>ED bar ratio $\rho_{ED}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BFRP bars ($E=48.4$ GPa, $\varepsilon_u=2.31%$)</td>
<td>0.0</td>
<td>0.5</td>
</tr>
<tr>
<td>CFRP bars ($E=140.0$ GPa, $\varepsilon_u=1.32%$)</td>
<td>0.6</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>0.8</td>
<td>2.0</td>
</tr>
</tbody>
</table>

Note: $E$=elastic modulus in tension of FRP bars; $\varepsilon_u$=ultimate strain in tension of FRP bars.

![Figure 4 Comparison of pushover curves of hybrid bars reinforced PTPS piers and ED bar reinforced ones](image)

**Pushover analysis results**

The aforesaid 32 specimens were firstly analysed under monotonic displacement. Due to limited space, Figure 4 only presents the pushover curves for the eight specimens of which the ED bar ratio $\rho_{ED}$ equalled to 1.0%. BFRP bars and CFRP bars were used in the specimens of Figure 4(a) and (b), respectively. The ultimate point in this study is defined as the drift ratio at which the FRP bars reached the ultimate tensile strain $\varepsilon_u$ and broke, or the drift ratio at which the lateral resistance degraded to 85% of the lateral load capacity. From this figure it is evident that
the FRP bars in the hybrid bars reinforced PTPS piers are very effective on increasing the lateral load capacity and the lateral stiffness after the specimens yield.

The influences of the FRP bars on the post-yield stiffness of the hybrid bars reinforced PTPS piers were further investigated. In this study, the post-yield stiffness ratio $r$ is defined in the following equation:

$$r = \frac{P_{\text{max}} - P_{\text{yield}}}{D_{\text{yield}}} \cdot \frac{D_{\text{yield}} - D_{\text{max}}}{P_{\text{max}} - P_{\text{yield}}} = P_{\text{max}} - P_{\text{yield}}$$

where, $P_{\text{max}}$ and $D_{\text{max}}$ are the lateral resistance and displacement at the peak point of the pushover cure, respectively; while the $P_{\text{yield}}$ and $D_{\text{yield}}$ are the lateral resistance and displacement, respectively, at the yield point defined by the energy method. The post-yield stiffness ratios for all the 32 specimens were shown in Figure 5.

BFRP bars and CFRP bars were adopted in the specimens in Figure 5(a) and (b), respectively. As indicated in this figure, adding additional FRP bars into ED bar reinforced PTPS piers is very effective on increasing the post-yield stiffness ratio. Furthermore, it can be found by comparing Figure 5(a) and (b) that, FRP bars with higher elastic modulus are more effective to improve the post-yield stiffness ratio. In addition, Figure 5 shows that the post-yield stiffness ratio of hybrid bars reinforced PTPS piers increases with the proportion of FRP bars to ED bars.

Cyclic analysis results

Residual drift ratio

Based on the pushover analysis results, the CFRP bars of all the hybrid bars reinforced PTPS pier specimens broke at drift ratios smaller than 4%. Therefore, only BFRP bars were used in the cyclic analysis. A total of 16 specimens were analysed under lateral cyclic displacement excursions up to 4% drift ratio with an increment of 0.5% drift ratio. In order to illustrate the differences in lateral cyclic behaviour between the hybrid bars reinforced PTPS piers and the ED bar reinforced ones, four specimens were chosen as examples and their hysteretic curves were plotted in Figure 6. It is evident from this figure that reasonable amount of FRP bars is very effective on decreasing the residual drift ratios of the PTPS piers. Here, the residual drift ratio is defined as the ratio of the residual displacement to the height of the PTPS pier.

In order to systematically investigate the effect of FRP bars on decreasing the residual drift ratio of hybrid bars reinforced PTPS piers, the residual drift ratio to the loading drift ratio curves for all 16 specimens were plotted in Figure 7. For the four specimens in each of the Figure 7(a) ~ (d), the ED bar ratios were identical while the FRP
bar ratios were variable. It can be drawn from the Figure 7 that if the ED bar ratio of a PTPS pier is no larger than 1.5%, reinforcing with additional FRP bars was very effective on reducing the residual drift ratios.

![Figure 7 Residual drift ratio of hybrid bars reinforced PTPS piers](image1)

**Cumulative hysteretic energy dissipation ability**

Cumulative energy dissipation is an important indicator of seismic performance of a structural member, as insufficient energy dissipation may lead to overlarge displacement demand of structures in earthquake disasters. Therefore, cumulative energy dissipation at the end of cyclic analysis $E_d$ of all specimens was calculated and plotted in Figure 8. The analysed specimens were separated into four group based on the ED bar ratio (ranging from 0.5% to 2.0%) and each group consisted four specimens with variable BFRP bar ratio (ranging from 0% to 0.8%). As shown in this figure, the cumulative energy dissipation $E_d$ for the four specimens in the same group was generally the same. This result indicates that the energy dissipation ability of the hybrid bars reinforced PTPS pier is comparable to its ED bar reinforced counterpart.

![Figure 8 Cumulative energy dissipation of hybrid bars reinforced PTPS piers](image2)

**CONCLUSIONS**

This paper presented a numerical analysis on the hybrid bars reinforced PTPS bridge pier, i.e., the pier reinforced with both FRP bars and steel bars. Based on the analysis results presented herein, the following conclusions can be drawn:

1. The FRP bars in the hybrid bars reinforced PTPS pier were very effective to improve the lateral load capacity and the post-yield stiffness ratio. This improvement would be even more evident if FRP bars with higher elastic modulus were used.
2. Increasing the proportion of FRP bars to ED bars would increase the post-yield stiffness ratio of hybrid bars reinforced PTPS piers.
Comparing to the ED bar reinforced PTPS pier counterparts, the hybrid bars reinforced piers exhibited smaller residual drift ratio and similar cumulative hysteretic energy dissipation.

The focus of the current study was to numerically validating the concept of hybrid bars reinforced PTPS piers. Further experimental validation and systematic analysis are also under way.

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REFERENCES


INVESTIGATION INTO THE BEHAVIOR OF REINFORCED CONCRETE BEAMS RETROFITTED WITH FRP FABRICS SUBJECTED TO BLAST LOADING

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2 Professor, Department of Civil and Environmental Engineering, Amirkabir University of Technology (Tehran Polytechnic), Tehran, Iran Email: mzkabir@aut.ac.ir
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ABSTRACT

Loads of explosion, either within or adjacent structures, have rapid transient nature and the potential to cause collateral damage and casualties could be considerable. Dynamic load at high strain rates, such as the explosion, changes in the mechanical properties of materials depends on the strain rate and the probability of failure mode from ductile flexural failure mode under static and quasi-static loading to brittle shear mode under dynamic load. Therefore, taking into account the considerations in the analysis and design of structures and components, providing safe and economic solutions for efficient operation of the structure and its components is important in their lifetime. In recent years, the use of fiber-reinforced polymers (FRP) is a common technique in order to retrofit and rehabilitation of reinforced concrete structures and other structures. In this paper, the behavior of reinforced concrete beams retrofitted with FRP fabrics subjected to blast loading is investigated. In this regard, first with the finite element method and the help of software modeling in LS-DYNA results in order to validate the results of the laboratory studies were compared. Then, with the development of numerical models the effect of parameters such as the type of retrofit, fiber, and explosive intensity, and reduce the use of FRP materials in order to strengthen economic plan, on response of beam in terms of displacement, crack pattern and failure mode, ductility and energy absorption is studied.

KEYWORDS

Reinforced concrete beam, Fiber Reinforced Polymers, blast loading, finite element method.

INTRODUCTION

Previous studies focused on flexural members of reinforced concrete under blast loading through experimental, analytical and numerical methods. Experimental studies in this field are limited due to high cost and other restraints. Concrete slabs reinforced with FRP composite layers experimentally examined under blast load. The results indicated that FRP-reinforcement can improve performance and expand the load bearing capacity of slabs under blast load (Razaqpur et al. 2007; Silva and Lu 2007; Kim et al. 2009; Tanapornraweekit et al. 2010; Orton et al. 2013). The behavior of concrete beams experimentally and numerically examined under blast loading. Concrete beams designed to fail in flexure may fail in a shear mode when loaded dynamically (Magnusson et al. 2010I, 2010II; Morales Alonso et al. 2013). Also, reinforced concrete beams analytically studied under blast load. The results indicated that using approximated and simplified methods such as a one-degree-of-freedom system can provide smaller amounts of stress than reality (Hamed and Rabinovitch 2005; Carta and Stochino 2013). In this paper, the control samples are constructed according to laboratory research data by Magnusson et al. (2010) and Tanapornraweekit et al. (2010). The numerical models are constructed based on finite element method through LS-DYNA software (2012). After validating the results of the numerical models, the behavior of reinforced concrete beams retrofitted with FRP fabrics subjected to blast loading is investigated.

Validation OF NUMERICAL MODEL

According to our literature review, no research was found on retrofitted RC beams by using FRP against blast loading. On the other hand, experimental research has been carried out on FRP retrofitted RC slabs. So, in this
paper, validation is limited to two parts. First, unstrengthened RC beams under blast load and second, strengthened RC slabs under blast loading. Table 1 shows definition of tested specimens.

Magnusson et al. (2010) experimentally assessed B40 beam under blast load. The beam dimensions and its reinforcement have been illustrated in Figure 1. Figure 2 shows pressure-time history that measured during the test.

![Figure 1 Dimensions and reinforcement of the concrete beam. Dimensions in mm](image)

![Figure 2 Reflected pressure applied in the analysis of the beam (Morales Alonso et al. 2013)](image)

![Figure 3 Dimensions and reinforcement of the concrete slab. Dimensions in mm](image)

Tana Pornraweekit et al. (2010) performed the laboratory research that involved an unreinforced C3 slab and reinforced G-2S-1L slab with a GFRP layer on both sides of the slab. The slab dimensions and its reinforcement have been illustrated in Figure 3. Blast loading applied in modeling is extracted from Conwep (2005) as a pressure-time curve derived from 0.45 Kg of TNT at a distance of 0.5 meters from the center of the slab.

The material model used in the software for simulation of concrete is concrete damage Model (072R3). Plastic-kinematic model is adopted for behavior of reinforcement steel. Moreover, composite layer involved composite material model 54 labeled Enhanced Composite Damage. Contact between the concrete and the composite layer is modeled through AUTOMATIC-SURFACE-TO-SURFACE-TIEBREAK. According to experimental data, the concrete compressive strength in the modeling is considered 43 MPa while the steel yield stress is 604 MPa. The concrete compressive strength in the slab modeling is considered 32 MPa while the steel yield stress is 356 MPa.

Figure 4 compares the crack pattern of B40 beam in the laboratory sample and numerical model. Comparison between the midspan displacement and beam support reaction modeled through laboratory samples have been illustrated in Figure 5. Furthermore, the comparison between the displacement results of laboratory and numerical modeling for unreinforced slab and slab reinforced with a composite layer has been shown in Figure 6. The slight difference in the results is due to such factors as uncertainty of behavior modeling and the damage to concrete and blast loading.

![Figure 4 Predicted and Experimental Crack Patterns for B40 Specimen (Magnusson et al. 2010)](image)
PARAMETRIC STUDY

In this paper, calibrated B40 beam in the previous section is retrofitted in flexural and shear terms in different ways and different types of fibers through FRP layers with a thickness of 0.5 mm. Table 2 describes the retrofitted samples, each examined through unidirectional layers of carbon fiber composite (CFRP), glass fiber (GFRP) and Kevlar fibers (KFRP). In the Stirrup Shaped specimens, the middle stirrup has 300 mm length and the other stirrup has 150 mm length in the free span of the beam. The mechanical properties of each unidirectional layer have been illustrated in Table 3.

Table 2 Specimen designations and their strengthening schemes

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Description of Strengthening</th>
<th>Type of Retrofitting</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer &amp; U-Shape End</td>
<td>The composite layer on the lower side and U-shaped end</td>
<td></td>
</tr>
<tr>
<td>U-Shape</td>
<td>U-shaped composite layer</td>
<td></td>
</tr>
<tr>
<td>Stirrup Shape</td>
<td>Stirrup-shaped composite layer</td>
<td></td>
</tr>
</tbody>
</table>

Maximum peak pressure, time duration, and impulse are the main characteristics of explosions. In this paper also, the intensity of the explosion is investigated. In this purpose, B40 beam is assessed exposure to five scenarios of blast loading with different peak pressure and time duration. For applying the same energy level on specimens the impulse of the explosion is assumed constant and it is equal to 3500 KPa-msec. Blast loading applied in modeling is extracted from Conwep as a pressure-time curve derived at a distance of 3.5, 5, 10, 15, and 20 meters from the explosion source. Figure 7 shows these curves. Then, for critical specimen according to table 2 effect of retrofitting is investigated.
Table 3 Mechanical Properties of FRP Composites

<table>
<thead>
<tr>
<th>Properties</th>
<th>Carbon/epoxy UD</th>
<th>E-glass/epoxy UD</th>
<th>Kevlar 49/epoxy UD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m³)</td>
<td>1580</td>
<td>1900</td>
<td>1400</td>
</tr>
<tr>
<td>Longitudinal modulus, $E_1$ (GPa)</td>
<td>138</td>
<td>40</td>
<td>75</td>
</tr>
<tr>
<td>Transverse modulus, $E_2$ (GPa)</td>
<td>9.65</td>
<td>8</td>
<td>6</td>
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<tr>
<td>In-Plane shear modulus, $G_{23}$ (GPa)</td>
<td>5.24</td>
<td>4</td>
<td>2</td>
</tr>
<tr>
<td>Out-of-plane shear modulus, $G_{23}$ (GPa)</td>
<td>2.24</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>Major poisson’s ratio, $\nu_{12}$</td>
<td>0.3</td>
<td>0.25</td>
<td>0.34</td>
</tr>
<tr>
<td>Longitudinal tensile strength, $X_t$ (MPa)</td>
<td>2280</td>
<td>1000</td>
<td>1300</td>
</tr>
<tr>
<td>Longitudinal compressive strength, $X_c$ (MPa)</td>
<td>1440</td>
<td>600</td>
<td>280</td>
</tr>
<tr>
<td>Transverse tensile strength, $Y_t$ (MPa)</td>
<td>57</td>
<td>30</td>
<td>30</td>
</tr>
<tr>
<td>Transverse compressive strength, $Y_c$ (MPa)</td>
<td>228</td>
<td>110</td>
<td>140</td>
</tr>
<tr>
<td>In-plane shear strength, $S$ (MPa)</td>
<td>71</td>
<td>40</td>
<td>60</td>
</tr>
</tbody>
</table>

Figure 7 Pressure-time curves at a distance of 3.5, 5, 10, 15, and 20 meters and impulse 3500 KPa-msec

RESULTS AND DISCUSSIONS

Ductility ratio is the ratio of the maximum displacement of the member to member yield displacement. The area under the force-displacement curve is elastic energy absorption until yielding of rebars, while the area under the force-displacement curve is maximum energy absorption until the maximum load, where If the elastic energy absorption is subtracted, plastic energy absorption will be obtained. The force-displacement curve for reference B40 beam and retrofitted beams have been shown in figures 8. For reference B40 beam the ductility ratio, maximum energy absorption, and plastic energy absorption is 9.82, 3040 J, and 2777 J, respectively. Figure 9 shows reduction of these criteria in retrofitted specimens in percent in comparison with B40 beam.

Several laboratory studies have shown that reinforced concrete members going through ductile flexural failure under quasi-static loading might experience brittle shear failure under dynamic loading (Magnusson et al. 2014). Therefore, cracking pattern is predicted in figure 10. According to figure 9, Percentage of the cracked height of retrofitted specimens have been shown in figures 11.
Assessing the intensity of the explosion on the B40 beam indicates that among different explosive loading conditions having the same energy level, the impulsive loading condition with less duration time and high pick pressure, is the most critical one in terms of beam displacement and crack growth (Figure 12(a) and Figure 13(a) ). In these figures, R is the distance from the explosion source in the meter. The effect of strengthening on the response of the beam in terms of crack pattern and displacement have been shown in figure 12 and 13.

CONCLUSIONS

Results show the maximum reduction of displacement in the case of U-shape retrofit mode with carbon fibers. Also, U-shape mode with carbon fibers leads to the most reductions in the growth of flexural cracks as well as flexural-shear cracks along the free span of the beam. Ductility of the beam is reduced in different states of retrofitting, which the greatest amount of ductility reduction is obtained when reinforcement with carbon fibers. Maximum and plastic energies absorbed by the retrofitted beams are decreased in all of the investigated cases. Retrofitting with carbon fibers shows the greatest reduction in the plastic energy absorption, in different states. The retrofit plan of the beam using stirrup shape made of carbon, kevlar or glass fibers leads to the optimum state of strengthening from the viewpoints of ductility and plastic energy absorption as well as Reduce consumption of FRP materials in comparison with the other plans.

REFERENCES

Performance in Severe Environments
ABSTRACT

This study aimed to reveal the strengthening effect of an externally bonded fiber reinforced polymer (FRP) sheet for deteriorated reinforced concrete (RC) members. The salt breeze damages RC structures located in coastal areas. Against this background, the authors prepared RC beams having a corroded main reinforcement bar. The corroded bars were simulated by electrolytic corrosion process. The target ratio of corrosion mass loss was 30%. An FRP sheet was then bonded to the lower side of the RC beam for flexural strengthening. Thereafter, the authors conducted a flexural loading test. The test results show that the maximum load capacity after strengthening using an FRP sheet was almost equal to the control beam’s capacity even if damaged cover concrete was unrecovered using section repair. However, the ductility of the beam could not be restored after strengthening. If the cover concrete was repaired, the performance of the flexural behavior after strengthening was recovered to be the same as the control beam’s performance.

KEYWORDS

Salt attack, FRP sheet, strengthening, cover concrete, section repair.

INTRODUCTION

Reinforced concrete (RC) structures located in coastal areas suffer from reinforcement corrosion due to the salt breeze. Strengthening techniques for reinforced concrete structures damaged by salt have been proposed by many researchers around the world. Against this background, this study will focus on a strengthening technique using a fiber reinforced polymer (FRP) sheet for an RC member damaged by salt. In order to restore an RC member deteriorated due to reinforcement corrosion, the corroded reinforcement bars can be strengthened using splice reinforcement, or the damaged cover concrete parts can be repaired using section padding. However, the cost and effort spent on strengthening and repair is considerable. Therefore, an administrator of a structure needs to decide on the maintenance priority for the RC structure while considering cost limitations. However, if the administrator can identify a strengthening technique to realize a short-term prolongation of the lifetime of the structure, it can be added as a new choice for maintenance. Thus, the authors will discuss the effect of a repaired or non-repaired section patch for strengthening using an FRP sheet bonded to an RC beam deteriorated by salt.

Table 1 Summary of the specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Strengthening</th>
<th>The ratio of corrosion mass loss(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Target value</td>
</tr>
<tr>
<td>No.1</td>
<td>None</td>
<td>0</td>
</tr>
<tr>
<td>No.2</td>
<td>None</td>
<td>34.9</td>
</tr>
<tr>
<td>No.3</td>
<td>CFRP sheet without section repair</td>
<td>30</td>
</tr>
<tr>
<td>No.4</td>
<td>CFRP sheet with section repair</td>
<td>40.1</td>
</tr>
</tbody>
</table>

OUTLINE OF THE EXPERIMENTS

All the specimens used in this study are summarized in Table 1. The parameter of the experiment was repair or non-repair of the cover concrete damaged by reinforcement bar corrosion. The FRP sheet was fabricated from carbon fiber (CFRP sheet).
Figure 1 A schematic view of the RC beam.

Table 2 The mixture proportion of concrete for the RC beam.

<table>
<thead>
<tr>
<th>W/C (%)</th>
<th>Air (%)</th>
<th>s/a (%)</th>
<th>Unit weight (kg/m³)</th>
<th>AE (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>45.2</td>
<td>5.5</td>
<td>36</td>
<td>149</td>
<td>673</td>
</tr>
</tbody>
</table>

The mixture proportion of concrete for the RC beam is summarized in Table 2. The water-cement ratio was 45%, and the mixed cement was high early strength cement. The compressive strength after a flexural loading test was 42.1 N/mm².

Electrolytic corrosion test

An electrolytic corrosion test was conducted on the RC beams, except for specimen No.1. A schematic view of the test is shown in Figure 2. According to the specifications of the common test, the specified current density was almost 10A/m² after starting conduction. According to that specification, the current density was determined to be 10A/m². An electric current of 0.65A was also applied during the test. To fulfill the target ratio of the corrosion mass loss, the current integration amount and the energizing time was calculated as follows (Tamori et al. 1988):

\[ W = 0.766 \sum (I \times T) \]  

where \( W \) is the amount of corrosion mass loss [g], \( I \) is the electric current [A], and \( T \) is the energizing time [hours].

In order to reproduce the damage of an RC member such as the deterioration period (JCI, 1998), the target corrosion mass loss ratio was set to 30%. The electrolytic corrosion test was conducted for three RC beams prior to strengthening using a CFRP sheet. The test period was approximately 1067 hours (44.5 days).
After the electrolytic corrosion test, deteriorated RC beams were taken out from the submersion tank. One week later, some deteriorated beams were strengthened using a CFRP sheet. The material properties of the CFRP sheets and the section repair material are shown in Tables 3 and 4, respectively. The fiber direction of the CFRP sheet was one-way. The section repair material was ordinary polymer cement mortar, and was a pre-mixed type. In order to realize flexural strengthening, the CFRP sheet was bonded underneath the RC beam. Specimen No.3 was bonded using a one-layer CFRP sheet. The damaged cover concrete of this specimen was not repaired. Details of strengthening method are as follows: 1. The surface of the beam’s bottom was treated with a grinder to remove laitance and corrosion products. 2. Then, the surface was primed with resin. 3. A CFRP sheet was bonded to the bottom of the beam between both supports. The bonded CFRP sheet was 80mm in width and 1220mm in length. In specimen No.3, corrosion cracks had a maximum width of 2.5mm and an average width of 0.7mm. However, in consideration of the convenience of construction work, the crack injection work was not conducted before

<table>
<thead>
<tr>
<th>Basis weight (g/m²)</th>
<th>Tensile strength (N/mm²)</th>
<th>Elastic modulus (kN/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>212</td>
<td>4420</td>
<td>252</td>
</tr>
</tbody>
</table>

Table 3 Material properties of the CFRP sheet.

<table>
<thead>
<tr>
<th>W/B</th>
<th>Compressive strength (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>13.4</td>
<td>7days</td>
</tr>
<tr>
<td></td>
<td>28days</td>
</tr>
<tr>
<td>35.3</td>
<td></td>
</tr>
<tr>
<td>48.2</td>
<td></td>
</tr>
</tbody>
</table>

Table 4 Material properties of section repairs.

(1) Chipping the damaged cover concrete, (2) Setting the specimen to the formwork, (3) Repairing the cover concrete by using section repair material

Figure 3 Repair method for damaged cover concrete.

**Strengthening using a CFRP sheet**

(1) Chipping the damaged cover concrete, (2) Setting the specimen to the formwork, (3) Repairing the cover concrete by using section repair material
strengthening. The damaged cover concrete was repaired using section repair material, as shown in Table 4. The repair method is shown in Figure 3. First, damaged cover concrete was carefully removed by hand using a small chipping hammer. Then, the beam was set to the formwork that was used during beam casting. After that, the beam surface was wetted after the removal of cover concrete. Then, section repair work was conducted. Finally, a CFRP sheet was bonded to the bottom of the beam, as with specimen No.3.

Figure 4 The distributions of the corroded reinforcement bar diameter.

Loading test

A flexural loading test was conducted for all specimens, as shown in Figure 1. The load, deflection at the mid-span, and strain of the sheet were measured using a data acquisition system. The sheet strains were measured every 70 mm.

Measurement of diameters of corroded reinforcement bars

In specimen Nos.2 to 4, the corroded reinforcement bars were excavated after the loading test. Then, the corrosion product was removed by soaking in an aqueous ammonium citrate solution. After that, in order to check the distribution of the corrosion mass loss, the decreased diameter of the reinforcement bar was measured every 25mm. The bi-directional diameters were measured in orthogonality.

RESULTS AND DISCUSSION

Corrosion behavior of reinforcement bars

The average corrosion mass loss ratio of the main reinforcement bar is shown in Table 1. The ratios for specimens Nos.3 and 4 were greater than the target ratio of 30%. The distributions of the corroded reinforcement bar diameter are shown in Figure 4. The non-corroded diameter is plotted in this figure. The diameter of specimen No.2 was unevenly distributed. The declining ratio of the diameter, which was more than 30%, was a maximum for specimen No.2.

Flexural behavior of loading test

The relationships between the load and deflection at the mid-span obtained from loading tests are shown in Figure 5. In addition, the strain distributions of the CFRP sheet are shown in Figure 6. The flexural behavior of specimen No.3, which did not have a repaired damaged cover concrete, is discussed as follows. The relationships for specimen Nos.1 and 2 are plotted in Figure 5. Specimen No.1 was a control specimen, and specimen No.2 had a corroded main reinforcement bar without repairs. Specimen No.1 exhibited flexural compression failure. On the other hand, specimen No.2 failed due to rupture at the corroded main reinforcement bar. The rupture location is indicated by the arrow shown in Figure 4. The bonded sheet of specimen No.3 de-bonded between the mid-span to the bonded end at the ultimate state. The maximum capacity of specimen No.3 was almost the same as that of specimen No. 1. However, the ductility of specimen No.3 decreased more than that of specimen No.1. This is
concerned about the distribution of the decreased diameter of the corroded main reinforcement bar. The diameter of specimen No.3 was greatly decreased near the location at the maximum flexural moment. As shown in Figure 6, the CFRP sheet strain increased near that location. The results of the above discussion suggest the possibility that the reduced ductility cannot be recovered by the strengthening process when the decreased cross-section of the main reinforcement bar is unevenly distributed.

![Figure 5 The relationships between the load and the deflection at the mid-span.](image)

![Figure 6 The strain distributions of the CFRP sheet.](image)

The load-deflection relationship of specimen No.4, which had repairs to the damaged cover concrete, is shown in Figure 5. Specimen No.4 had a bonded CFRP sheet subjected to flexural strengthening. This specimen failed due to sheet de-bonding at the ultimate state. The maximum capacity of specimen No.4 was greater than that of specimen No.1 because the damaged cover concrete was repaired prior to sheet bonding. In addition, the sheet strain of specimen No.4 is shown in Figure 6. The sheet strain increased at the ultimate state. Therefore, the tensile stress of the main reinforcement bar was redistributed to the FRP sheet.

**CONCLUSIONS**

The conclusions of this study are as follows:
1. Reinforced concrete beam specimens, which had a corroded main reinforcement bar, were fabricated. The ratio of corrosion mass loss was almost 30%.
2. The deteriorated beams were strengthened by bonding using a carbon fiber reinforced polymer (CFRP) sheet without repairing the damaged cover concrete. As a result, the ultimate flexural capacity was almost the same as
that of the control specimen. However, the ductility of the retrofitted specimen was not recovered because the cover concrete was still damaged.

3. On the other hand, another deteriorated beam was strengthened by bonding using a CFRP sheet with patch repair of the damaged cover concrete. The ultimate capacity after retrofitting was increased to be greater than that of the control beam.

The results of the above studies suggest that if damaged concrete is not repaired prior to FRP sheet bonding, the ultimate capacity after retrofitting will be almost the same as the initial capacity. However, if the damaged concrete is repaired, a strengthening effect will be achieved. However, the suggestions are based on small-scale beam test. In future work, the author will conduct the large-scale beam test, and then the detail of strengthening effect will be discussed.

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ABSTRACT

This paper presents the effectiveness of carbon fiber reinforced polymer (CFRP) sheets on protecting concrete in an acidic service environment. Concrete members constructed in an industrial zone frequently deteriorate and, accordingly, the need for strengthening arises. Concrete cylinders are prepared with and without CFRP-wrapping and are subjected to a 5% concentration of sulfuric acid (H$_2$SO$_4$) for six weeks. Interests include changes in concrete properties owing to the H$_2$SO$_4$ exposure, and the performance of CFRP-wrapped specimens. Various experimental approaches are employed: microscopic imaging, thermogravimetric analysis, infrared spectroscopy, and mechanical loading to failure. CFRP-wrapping retards the chemical interaction between the core concrete and H$_2$SO$_4$, which is active in the case of unstrengthened concrete. According to microscopic examinations, the plain concrete specimens are damaged by H$_2$SO$_4$ at its surface level; however, the CFRP-wrapped concrete is not degraded. This observation is supported by the thermogravimetric analysis in terms of a marginal mass reduction in the concrete with CFRP-wrapping. The strength of the plain concrete decreases by 57% due to the six-week conditioning, which is significant when compared with the strength drop of 23% observed in its CFRP-wrapped counterparts.

KEYWORDS

Axial, CFRP, damage, durability, environmental distress, strengthening, sulfuric acid (H$_2$SO$_4$).

INTRODUCTION

Industrial structures are frequently constructed with concrete. The presence of biogenic bacteria in such a service condition may degrade the performance of constructed members. Sulfuric acid (H$_2$SO$_4$) is widely observed in wastewater plants and sewer structures (Fattuhui and Hughes 1988; Vincke et al. 1999; Idriss et al. 2001). Equation 1 shows the chemical reaction that occurs when concrete interacts with H$_2$SO$_4$, creating gypsum and ettringite:

$$Ca(OH)_2 + H_2SO_4 \rightarrow CaSO_4 + 2H_2O$$

In the United States, an annual budget of $25 billion is required to operate and maintain wastewater infrastructure (O’Connell et al. 2010). It is also estimated that over $390 billion should be spent to repair deteriorated wastewater systems until 2030 (Gutierrez-Padilla et al. 2010). Because of this practical significance on acid-induced damage, active research has been conducted. Durning and Hicks (1991) examined the behavior of concrete subjected to several detrimental chemicals, namely, acetic acid (CH$_3$COOH), sulfuric acid (H$_2$SO$_4$), formic acid (HCO$_2$H), and phosphoric acid (H$_3$PO$_4$). The calcium silicate hydrate paste of concrete led to an increase in durability under these chemical exposure circumstances. Bassuini and Nehdi (2007) studied the effect of sulfuric acid on the performance of self-consolidating concrete. Differential scanning calorimetry and X-ray power diffraction were used. The integrity of the concrete was enhanced by adding steel and polypropylene fibers. The mass loss of concrete was not directly related to the loss of its load-bearing capacity. Monteny et al. (2003) cast concrete with various admixtures (e.g., superplasticizers, blast furnace cement, polymer, and silica fume), and exposed the concrete specimens to H$_2$SO$_4$. The performance of the concrete mixed with blast furnace cement was the best, whereas that of the concrete with silica fume was the worst.

Despite a wide variety of research endeavors, effective methods are still incomplete to extend the service life of constructed concrete structures in industrial regions. It appears that adding admixtures may have limited efficacy, since calcium hydroxide in concrete chemically reacts with H$_2$SO$_4$. A fundamentally different approach may
resolve current concerns. This paper discusses the potential of CFRP sheets to upgrade the capacity of constructed concrete members suffering sulfate-induced damage and, at the same time, protects the concrete, so that the members’ service life can be improved. An experimental program is conducted to chemically and mechanically examine the behavior of CFRP-wrapped concrete immersed in an H$_2$SO$_4$ solution.

**RESEARCH SIGNIFICANCE**

Sulfate-induced damage is a critical problem in industrial structures (e.g., concrete columns are directly exposed to chemicals in a sewer system). As such, constructed members deteriorate and need rehabilitation. CFRP strengthening is a proven technology with numerous advantages. Existing research encompasses a wide range of scopes from mechanics-oriented investigations to durability observations. To the best knowledge of the authors, no research has been conducted to evaluate the performance of CFRP-confined concrete when exposed to acidic environments. Given that CFRP-strengthening can likely be used for industrial structures, this research gap needs to be filled.

**EXPERIMENTAL PROGRAM**

**Materials**

Twelve concrete cylinders (100 mm in diameter and 200 mm in height) were cast in the laboratory to have a specified compressive strength of 20 MPa: Type III cement (500 g), water (250 g), fine aggregate (811 g), and coarse aggregate (1,700 g) per cylinder. Upon completion of moisture curing, six cylinders were wrapped with CFRP sheets. The CFRP has a nominal tensile strength of 3,800 MPa with a modulus of 227 GPa, based on its equivalent fiber thickness of 0.165 mm. The bonding agent used was a two-part epoxy, consisting of a hardener and a resin to be mixed at a ratio of 1:3 by weight. CFRP and epoxy coupons were prepared to examine the effect of H$_2$SO$_4$ exposure (to be explained). According to the manufacturer’s guidelines, the CFRP-wrapped cylinders were cured for seven days. The top and bottom surfaces of each cylinder were sealed using the epoxy to avoid direct exposure to H$_2$SO$_4$ (Figure 1).

**Chemical conditioning**

A 5% concentration H$_2$SO$_4$ solution was made in the laboratory, which is frequently used in accelerated durability testing (McMurry 2012). Two test categories were planned (i.e., plain concrete and CFRP-wrapped concrete). The cylinders and coupons were immersed in the solution, as shown in Figure 1. A pH meter was used to measure the acidity of the solution for six weeks. After achieving the planned six-week exposure period, the specimens were completely rinsed and dried for mechanical testing.

**Mechanical testing**

Two universal testing machines (90 kN and 900 kN capacities) were employed to load the coupons and cylinders, as shown in Figure 2. (loading rates of 2.5 mm/min and 5 mm/min for the coupons and cylinders, respectively). The deformation of these specimens was measured using a non-contact extensometer [Figure 2(a)].

![Figure 1 Test specimens: (a) CFRP and epoxy coupons; (b) prepared cylinders; (c) plain concrete in H$_2$SO$_4$; (d) CFRP-wrapped concrete in H$_2$SO$_4$.](image-url)
Chemical testing

Fourier transform infrared spectroscopy was used to examine the chemical interaction between H$_2$SO$_4$ and the CFRP and epoxy. Wave numbers ranged from 550 cm$^{-1}$ to 4000 cm$^{-1}$, and the specimens’ absorbance was detected at zero and six weeks of exposure. Thermogravimetric analysis was also conducted at elevated temperature from 20°C to 1,000°C to evaluate the decomposition of constituents in the concrete.

RESULTS AND DISCUSSIONS

Concrete deterioration

As the exposure time increased to six weeks, the pH values of the solution increased up to 64% and 197% for the CFRP-wrapped concrete and the plain concrete, respectively. This fact indicates that the plain concrete interacted with H$_2$SO$_4$, whereas the CFRP retarded the interaction between the core concrete and the acid. The plain cylinders were noticeably deteriorated, as shown in Figures 3(a) and (b). By contrast, the CFRP-wrapped concrete merely revealed a change in color [Figures 3(c) and (d)] without damage of the core. The mass-loss of these two test categories was 1% and 7% for the CFRP-wrapped and plain concrete specimens, respectively.

Chemical analysis

Figure 4 shows the results of the Fourier transform infrared spectroscopy for the epoxy and CFRP materials exposed to H$_2$SO$_4$. Although there was no remarkable change in chemical responses (i.e., functional groups), their absorbance was distinct. The C-H stretch of alkanes in the epoxy and CFRP (wavenumber 2850-3000 cm$^{-1}$) increased by 47% and 26% due to the exposure, respectively. The H-C==O:C-H stretch (wavenumber 2695-2830 cm$^{-1}$) increased by 22% in the epoxy and 13% in the CFRP. The -C=C- stretch of alkenes (wavenumber 1620-1680 cm$^{-1}$) increased by 20% and 13% in the epoxy and CFRP specimens, respectively. Other responses such as -C≡X−H: C-H bend of alkynes (wavenumber 610-700 cm$^{-1}$) were also noticeable.

Figure 3 Deterioration of specimens: (a) plain concrete at zero week; (b) plain concrete at six weeks; (c) CFRP-wrapped concrete at zero week; (d) CFRP-wrapped concrete at six weeks
Shown in Figure 5 are the results of the thermogravimetric analysis. The epoxy component in the CFRP composite disappeared at 420°C [Figure 5(a)]. The mass loss of the plain concrete exposed to H₂SO₄ was significant [Figure 5(b)], which is different from the case of the CFRP-wrapped concrete [Figure 5(c)]. These observations corroborate the fact that CFRP-wrapping protected the core concrete, although some acid penetration has occurred through the CFRP.

**Strength variation**

Figure 6 summarizes the capacity of the epoxy, CFRP, and concrete specimens before and after the H₂SO₄ exposure. The capacity of the epoxy was reduced by 60% [Figure 6(a)], whereas that of the CFRP decreased by 23% [Figure 6(b)]. This means that carbon fibers inside the composite played an important role in preserving its strength. At zero-week exposure, the confined concrete showed a 440% higher strength compared with the plain concrete [Figure 6(c)]. This trend was maintained at six weeks with a 600% increase in strength.

**Failure mode**

The failure modes of the concrete specimens exposed to H₂SO₄ for six weeks are illustrated in Figure 7. The failure of the plain concrete occurred owing to the development of a vertical splitting crack [Figure 7(a)], which is different from conventional concrete failure. This reveals that the cohesion characteristics of the cement binder were deteriorated by the acidic exposure. The strengthened concrete failed by typical CFRP rupture [Figure 7(b)], without showing any damage in the core concrete according to visual examinations.
Figure 6 Strength variation induced by H$_2$SO$_4$ exposure: (a) epoxy; (b) CFRP; (c) concrete

Figure 7 Failure mode: (a) plain concrete at six-week exposure; (b) CFRP-wrapped concrete at six-week exposure

CONCLUSIONS

This paper has dealt with the performance of CFRP-wrapped concrete subjected to H$_2$SO$_4$, with an emphasis on chemical and mechanical responses. Twelve concrete cylinders (six plain and six CFRP-wrapped specimens), and epoxy and CFRP coupons were immersed in a 5% concentration H$_2$SO$_4$ solution for six weeks, and tested. Below are conclusions from this experimental study:

- The degree of acidity was influenced by the interaction between the plain concrete and H$_2$SO$_4$, in conjunction with a mass loss of 7%. The CFRP-wrapped concrete, however, did not show a noticeable change in pH, which indicated that the CFRP protected the core concrete from the chemical interaction with H$_2$SO$_4$.
- The functional groups of the epoxy and CFRP were not influenced by the acid exposure. Nonetheless, their absorbance degree was altered by H$_2$SO$_4$. According to the thermogravimetric analysis, the integrity of the concrete protected by the CFRP was better maintained in comparison with that of the plain concrete.
- The load-bearing capacity of the epoxy and CFRP was reduced by 57% and 23% due to the exposure, respectively. The capacity of the CFRP-wrapped concrete was 600% and 400% higher relative to the plain concrete specimens at six weeks of exposure to H$_2$SO$_4$. The core concrete protected by the CFRP was undamaged, unlike the case of the plain concrete.

ACKNOWLEDGMENTS

The authors gratefully acknowledge financial support from the University of Colorado Denver. Ms. Catherine Rathbun has provided valuable advice for the chemical analysis presented herein.

REFERENCES


FLEXURAL BEHAVIOR OF CFRP-REINFORCED CONCRETE BEAMS ATTACKED BY ACID RAIN

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ABSTRACT

This paper studies the flexural behavior of Carbon Fiber-Reinforced Polymer (CFRP)-reinforced concrete beams attacked by acid rain. To this end, an artificial acid rain with a pH level of 1.5 was prepared by mixing sulfate and nitric acid solutions. Three reinforced concrete beams with CFRP applications were constructed and conditioned using the artificial acid rain under different corrosion time (0, 45, 90 days). After conditioning, the concrete strength and corrosion depth were measured by combined ultrasonic-rebound methods. Constitutive relationships of concrete under acidic corrosion were determined. Dynamic tests were applied to estimate the integrity of the beams. Four-point bending tests were performed to investigate the load capacities. The cracks on beams were recorded to analyze their effect on the flexural behavior of CFRP-reinforced concrete beams. Finite element (FE) models were built. The frequency, load-displacement curve, strain and the ultimate strength from FE models were correlated with test results. Both the test and FE results indicate that the failure mode of CFRP-reinforced concrete beam changes from balanced-reinforced beam failure to ductile failure as corrosion time increases.

KEYWORDS

Flexural behaviour, carbon fiber-reinforced polymer, reinforced concrete beams, acid rain, experimental investigation, finite element analysis.

INTRODUCTION

The properties of concrete structures degrade under aggressive environment such as acid rain, which typically has a pH value less than 5.7 (Larssen et al. 1999; Fan et al. 2014; Ernest et al. 2013). At least one fourth of the territory of North America is covered by acid rain (Varma et al. 1990). For example, a rain fall with a pH value of 1.5 was recorded in West Virginia (Keller 2012). The damage caused by acid rain is related to sulfate corrosion and expansive corrosion reaction when sulfide ions are combined with hydrogen ions, resulting in losses of billions of dollars per year in North America (Mansfeld et al. 1988). Therefore, it is necessary to study the mechanism of acid rain attack on concrete structures to minimize its impact. Fick’s II Diffusion Law is typically used for modeling concrete corrosion under acid rain (Tsutomu et al. 2001).

Composite materials such Carbon Fiber-Reinforced Polymer (CFRP) have been widely used with concrete structures over the last two decades due to their high tensile strength, light weight, durability, flexibility, corrosion resistance and ease of installation (Liu et al. 2014; Sen et al. 2015; Ali et al. 2012). Similar to other structures, CFRP-reinforced concrete structures are also attacked by acid rain during their service life (Francois et al. 2013). However, in contrast to extensive studies on reinforced concrete structure, studies on the damage mechanism and process, and loading capacities of CFRP-reinforced concrete structures attacked by acid rain are limited (Haber et al. 2012). Therefore, the degradation mechanism of CFRP-reinforced concrete structures under acid rain needs be investigated, which is the motivation of this study.

In this study, an artificial acid solution with a pH level of 1.5 was prepared by mixing sulfate and nitric acid solutions to corrode the reinforced concrete beams with CFRP applications. Bending load was imposed to the top surface of concrete beams to simulate the coupled acid-load action during conditioning. The concrete strength was
measured by ultrasonic-rebound combined method. Four-point bending tests were performed to investigate the load capacities, including cracking, yielding and ultimate loads; yielding and ultimate deflections; ductility; failure mode; and crack patterns. Finite element models were built. The load-displacement curve, strain and ultimate strength of concrete beams from FE model were compared with those from test results.

**EXPERIMENTAL INVESTIGATION**

**Design of specimens**

The dimension of concrete beams was 1800×120×200 mm$^3$ (Figure 1). Commercial concrete (C40) was used, with mix details shown in Table 1. Longitudinal steels were HRB400 (φ16) hot rolled reinforcing steel, with tensile strengths of 330 MPa. Stirrups were HPB335 (φ6). The concrete cover was 35 mm, a tensile strength of 3400 MPa and an elastic modulus of 220 GPa. The monolayer CFRP was attached to the bottom surface of the concrete beams with u-shaped clamp hoops on both sides (ACI 440. 2R-08, 2008), as shown in Figure 2.

<table>
<thead>
<tr>
<th>Table 1 Mix proportion of the concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (kg/m$^3$)</td>
</tr>
<tr>
<td>-------------------</td>
</tr>
<tr>
<td>450</td>
</tr>
</tbody>
</table>

Figure 1 Configuration of the specimens (unit: mm)

Figure 2 Anchoring scheme (unit: mm)

Figure 3 Concrete beams under acid environment


Figure 4 Loading setup (unit: mm)
Experimental procedures

The artificial acid rain was prepared in the laboratory to represent the chemical components of acid rain found in China, with a pH value of 1.5 (Hou et al. 2012). The molar ratio of sulfuric and nitric acid was 9:1. Since neutralization reaction occurred when Ca(OH)$_2$ inside concrete reacted with acid rain and nitric acid was volatile, the injection of nitric acid every 8 hours was needed to maintain a constant pH level. Based on relevant standards (ACI 343R-95, 1995), 1% of the ultimate capacity ($P_u$) of concrete beams was added to the top surface as an external bending load, which is shown in Figure 3. Table 2 lists details of experimental conditions.

<table>
<thead>
<tr>
<th>No.</th>
<th>Dimension (mm$^3$)</th>
<th>Corrosion environment</th>
<th>Corrosion time (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1800x120x200</td>
<td>Natural</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>H$_2$SO$_4$+HNO$_3$ (molar ratio 9:1)</td>
<td>45</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>(molar ratio 9:1)</td>
<td>90</td>
</tr>
</tbody>
</table>

After the artificial corrosion process was completed, comprehensive ultrasonic-rebound tests were conducted to evaluate the compressive strength of the concrete. Subsequently, an acceleration transducer was attached to the top surface of the beams. Based on the principle of wave propagation, a knock response method was adopted to measure the frequency of the beam. Next, four-point bending tests were performed to investigate the flexural behavior of the damaged beams (ACI 437.2-13, 2013). The experimental setup is shown in Figure 4.

Results and analysis

Test results are listed in Table 3. The relationship between load and deflection of CFRP-reinforced concrete beams are shown in Figure 8. Compressive strengths of concrete decrease as corrosion time increases, where the compressive strengths under 45 and 90 days are 34.1% and 25.3 % lower than that of non-corroded concrete. The frequency of U-3 is 22.9% larger than that of U-1, which indicates that the frequency of CFRP-reinforced concrete beam increases as corrosion time increases. It is similar to our previous results (Fan et al. 2014). This phenomenon can be attributed to the Hydrated Calcium Sulphate (H-C-S) which is the production of sulfate and expansive corrosion when sulfate ions penetrate into cementitious materials. The ultimate strengths of U-2 and U-3 are 4.0% and 6.1% lower than that of U-1, which manifests that the ultimate strength of CFRP-reinforced concrete beam decreases with corrosion time increases. The cracks under different loading levels and the failure mode of the CFRP-reinforced concrete beams are shown in Figure 5. The failure mode of U-1 and U-2 is a balanced-reinforced beam failure, which indicates that CFRP sheet in tensile zone is debonded and concrete in compressive zone is crushed almost at the same time. However, the failure mode of U-3 is more ductile, where concrete in the compressive zone is not crushed when CFRP sheet in tensile zone is debonded.

<table>
<thead>
<tr>
<th>No.</th>
<th>Compressive strength (MPa)</th>
<th>Frequency (Hz)</th>
<th>Pcr (kN)</th>
<th>Py (kN)</th>
<th>Pu (kN)</th>
<th>$\Delta y$ (mm)</th>
<th>$\Delta u$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>U-1</td>
<td>38.72</td>
<td>71.77</td>
<td>25</td>
<td>108.8</td>
<td>136.3</td>
<td>5.6</td>
<td>13.6</td>
</tr>
<tr>
<td>U-2</td>
<td>34.93</td>
<td>74.71</td>
<td>16</td>
<td>104.8</td>
<td>130.9</td>
<td>6.7</td>
<td>16.4</td>
</tr>
<tr>
<td>U-3</td>
<td>34.71</td>
<td>93.75</td>
<td>35</td>
<td>110.0</td>
<td>128.0</td>
<td>6.1</td>
<td>13.0</td>
</tr>
</tbody>
</table>

Figure 5 Cracks of beams under ultimate loads
FINITE ELEMENT ANALYSIS

Constitutive relationships

Mechanical properties of concrete, rebar and CFRP are listed in Table 4. Based on the concentration gradient between the interior and exterior of the concrete samples, $\text{SO}_4^{2-}$ spreads inside the concrete under acid rain. The function of concrete strength at different corrosion time can be expressed as:

$$f_e(t) = f_0 - k f_0 \alpha (t - t_0)^{-3} \sum_{n=1}^{\infty} \frac{(-1)^n}{n^3} \frac{n \pi}{R} \sin \left( \frac{e R}{R} \right) \left( \frac{-D n^2 \pi^2 t}{R^2} - e \frac{R^2}{R^2} \right)$$

where $f_e(t)$ is the strength of concrete at $t$ days under acidic corrosion, $f_0$ is the initial strength of concrete, $t_0$ is the day when the strength of concrete is the maximum, $k$ is the strength reduction ratio of concrete, $D$ is the damage parameter, and $R$ is the radius of concrete sample. A concrete damaged plasticity model is adopted to simulate cracking propagation until the failure of concrete beams. The yield function is:

$$F(\bar{\sigma}, \hat{\varepsilon}) = \frac{1}{1-\alpha} (\bar{q} - 3 \alpha \bar{p} + \beta(\hat{\varepsilon}^{pl}) \hat{\sigma}_{ms} - \gamma(-\hat{\sigma}_{ms})) - \sigma_{pl}^{t} \leq 0$$

where $\alpha$, $\beta$ and $\gamma$ are dimensionless material constants; $\hat{\sigma}$, and $\hat{\sigma}$ are effective tensile and compressive cohesion stresses, respectively; $\bar{q}$ is the Mises equivalent effective stress; $\bar{p}$ is the effective hydrostatic pressures; and $\hat{\varepsilon}^{pl}$ is strain function which is defined as:

$$\beta(\hat{\varepsilon}^{pl}) = \frac{\bar{\sigma}}{\hat{\sigma}} \frac{(\hat{\varepsilon}^{pl})}{(1-\alpha) - (1+\alpha)}$$

In this study, we assume that the damage only occurs in concrete. The strength of concrete attacked by the acidic corrosion is from comprehensive ultrasonic-rebound tests above.

<table>
<thead>
<tr>
<th>Materials</th>
<th>Elasticity modulus (GPa)</th>
<th>Poisson's ratio</th>
<th>Yield stress (MPa)</th>
<th>Yield strain (%)</th>
<th>Ultimate tensile stress (MPa)</th>
<th>Ultimate tensile strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>32.5</td>
<td>0.22</td>
<td>-</td>
<td>-</td>
<td>1.150</td>
<td>0.010</td>
</tr>
<tr>
<td>Longitudinal rebar</td>
<td>210</td>
<td>0.30</td>
<td>300</td>
<td>0</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>CFRP</td>
<td>70.0</td>
<td>0.10</td>
<td>-</td>
<td>-</td>
<td>600</td>
<td>0.85</td>
</tr>
</tbody>
</table>

FEA models

The geometry of the Finite Element (FE) model is shown in Figure 6. The Model has 14400 CPS4R, 1200 S4R elements and 1200 T3D2 elements to represent concrete beam, CFRP sheets and longitudinal rebar, respectively. A displacement was applied on the top of the model.

![Figure 6 FE model](Image)

Results and comparisons

Based on the FE analysis, the frequency and mode of vibration under different modes are obtained. Comparisons of frequency between test and FE results are shown in Figure 7. The load-deflection curves of test and modeling results under different corrosion times are shown in Figure 8. The cracks and plastic strain of CFRP-reinforced concrete beams from both test and modeling results under different corrosion times are shown in Figure 9.

The frequency of the FE model increases as corrosion time increases, which is similar to the test results. However, the frequency of model corroded by 90 days is only 3.3% larger than that of non-corroded model. The load-deflection curves from test and FE results closely correlated with each other, where the ultimate strength of the FE
model decreases with corrosion time increases. The effective plastic strain of the FE model is shown in Figure 9, which represents cracks of concrete beam under compressive load. The result shows that cracks are generated in tensile zone and concrete in compressive zone is crushed, which indicates that failure mode of non-corroded model is balanced-reinforced beam failure. From Figure 9(b), it can be seen that some cracks along the loading direction are obvious in the tensile zone of the FE model. However, crack is not generated in the compressive zone, which indicates that concrete in compressive zone is not crushed. Similar to U-3, the failure mode of the corroded model (90 days) is more ductile.

CONCLUSIONS

From this study, the following conclusions can be drawn:
(1) The frequency of the beam under 90 days’ corrosion is 22.9% larger than that of non-corroded beam, which indicates that the frequency of CFRP-reinforced concrete beam increases as corrosion time increases. The ultimate strengths of beams under 45 and 90 days’ corrosion are 4.0% and 6.1% lower than that of non-corroded beam, which manifests that the ultimate strength of CFRP-reinforced concrete beam decreases with corrosion time increases.
(2) The failure mode of CFRP-reinforced concrete beam become more ductile (corroded by 90 days) compared to the balanced-reinforced beam failure (corroded by 0 and 45 days), which indicates that long-term acidic corrosion has negative effect on the ultimate bending capacity of CFRP-reinforced concrete beam.
(3) Based on the strength from test, constitutive relationships of different materials are determined. The flexural behavior of FE model is similar to the test results. Similar to the test results, the failure mode of non-corroded model is a balanced-reinforced failure. However, corroded model is more ductile.

ACKNOWLEDGEMENTS

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ABSTRACT

An experimental study was conducted to investigate the axial compressive performance of CFRP-confined concrete after exposure to a cyclic wet-dry environment with 3.5% salt solution. A total of sixty 150x300mm concrete cylinders unconfined or confined with CFRP were tested. The test parameters included the number of wet-dry cycles (0, 60, 120, 240 and 360) and the number of CFRP layers (0, 1, 2 and 3). The results show that the axial compressive strength and hoop strain of concrete confined with three layers of CFRP were reduced by up to 13% and 19% respectively after 360 wet-dry cycles. The ultimate axial strain increased with the increase of the number of CFRP layers, but the effect of the number of wet-dry cycles was insignificant.

KEYWORDS
Confined concrete, durability, CFRP, wet-dry cycle, compressive performance.

INTRODUCTION

Due to aging of existing infrastructures and an increase of natural disasters such as earthquake, there has been an increasing demand to rehabilitate or strengthen damaged/deteriorated concrete structures (Guo et al. 2016). Because of their high strength-to-weight ratios and superior corrosion resistance, fibre reinforced polymer (FRP) composites have been widely used for this purpose. In particular, the use of FRP wrapping has been a very popular research topic for confining concrete to improve its compressive strength and ductility. However, studies on the long-term behaviour of FRP-confined concrete are relatively scarce, which could significantly hinder the application of FRP in structural reinforcement (e.g. Micelli et al. 2015; Eldridge et al. 2014). Hanna and Jones (1997) tested and studied how the compressive strength of glass FRP (GFRP) confined concrete was affected by environmental factors, including humidity, saline environments and different temperature cycles. The effect of wet-dry cycles on the compressive strength of carbon FRP (CFRP) confined concrete have also been studied by a number of researchers (e.g. Toutanji 1999; Toutanji and Deng 2002; Gharachorloou and Ramezanianpour 2010). In addition, Belarbi et al. (2004) and Belarbi and Bae (2007) investigated the effects of freezing-thaw cycles, high-temperature cycles, high-humidity cycles, ultraviolet (UV) radiation and immersion in saline solution on the compressive strength of GFRP and CFRP confined concrete. These tests showed that these environmental factors reduce the compressive behaviour of FRP confined concrete. However, more research is required because the conclusions were drawn based on limited research so are rather qualitative. In particular, the compressive behaviour of CFRP-wraped concrete exposed to cyclic wet-dry environment is still not clear.

In this study, five series of specimens were tested to investigate the compressive behaviour of CFRP-confined concrete after exposing to a cyclic wet-dry environment.

EXPERIMENTAL PROGRAMME

Test specimens and materials
A total of 60 cylinder specimens of 150 mm diameter and 300 mm height were cast. They were divided into 5 series, subjected to 0, 60, 120, 240 and 360 wet-dry cycles respectively. A saline solution with 3.5% salt was used. Each cycle took 24 hours: 16 hour wet and 8 hour dry. The temperature was set at 40°C throughout the wet-dry process. Each series of specimens consisted of four groups of three specimens wrapped with 0, 1, 2 and 3 layers of CFRP respectively. All specimens subjected to the cyclic wet-dry environment had both top and bottom faces sealed with epoxy. All cyclic tests started when the concrete was 28 day old.
The mix proportion and cylinder compressive strength of concrete are shown in Table 1. The CFRP used in this study was UT70-30 unidirectional carbon fibre sheet with a nominal thickness of 0.167 mm for each layer. The stress-strain curves for both CFRP and epoxy resin were tested with seven flat coupons. The results are shown in Figures 1(a) and 1(b) respectively, where CFRP specimens are designated by CRT-n and epoxy specimens designated b ERT-n, here n is the specimen number. The CFRP had a tensile strength of 4569 MPa and a Young’s modulus of 232.0 GPa. The epoxy resin had a tensile strength of 35.5 MPa and a Young’s modulus of 2.48 GPa.

<table>
<thead>
<tr>
<th>Cement (kg/m³)</th>
<th>Water (kg/m³)</th>
<th>Sand (kg/m³)</th>
<th>Gravel (kg/m³)</th>
<th>W/C</th>
<th>Sand ratio</th>
<th>Slump (mm)</th>
<th>Cylinder compressive strength (28 days, MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>440</td>
<td>242</td>
<td>670</td>
<td>1092</td>
<td>0.55</td>
<td>0.38</td>
<td>80</td>
<td>47.3</td>
</tr>
</tbody>
</table>

![Figure 1](image1.png)

### Instrumentation and testing

The specimens were tested under uniaxial compression in an electro-hydraulic servo test machine MATEST5000 with a capacity of 5000 kN. They were all tested under displacement control with a cross-head displacement rate of 0.18 mm/min. The longitudinal deformation in the middle third of the specimens was measured using two LVDTs. Four 20 mm foil strain gauges were evenly placed around the specimen at the mid-height to measure the hoop strain, as shown in Figure 2.

![Figure 2](image2.png)
**EXPERIMENTAL RESULTS AND DISCUSSIONS**

**Stress–strain relationship**

Figure 3(a) shows the stress-strain curves of the unconfined cylinder specimens after exposure to different numbers of wet-dry cycles. Each specimen is designated by either RT-n-m or WD-n-m, where RT stands for room temperature indoor environment and DW stands for cyclic wet-dry environment, n is the number of CFRP layers and m is the number of wet-dry cycles. “-A” or “-H” are added at the end of the specimen designations, with the former representing axial strain and the latter horizontal (circumferential or hoop) strain. It shows that comparing to the RT specimens the concrete strength initially increased slightly with an increase of wet-dry cycles. As the strength of the RT specimens were measured at 28 days, this should be interpreted that the increase of the strength of concrete was faster due to the increase of age than the reduction due to the effect of wet-dry cycles. When the number of wet-dry cycles continued to increase, the strength slightly reduced as the increase due to aging became less significant.

Figure 3(b), (c) and (d) show the stress-strain curves of CFRP-confined concrete exposed to different wet-dry cycles. Clearly, all the CFRP strengthened specimens had nearly bi-linear stress-strain curves. For specimens strengthened with one layer of CFRP (Figure 3(b)), the number of wet-dry cycles had almost no effects on the stress versus both axial and circumferential strains when the stress is lower (before the CFRP is activated at about 48 MPa). After that, the slope of stress versus both axial and hoop strain curves of the confined concrete gradually decreased with an increase of the exposure cycles (Figure 3(b)). When the CFRP confinement was increased to 2 or 3 layers, the slope of the hardening (second) branch reduced with an increase of the number of wet-dry cycles (Figure 3(c) and (d)).
After 360 wet-dry cycles, the compressive strength of the unconfined cylinders was increased by 9.4% but the ultimate axial strain was reduced by 9.4% and the ultimate hoop strain was reduced by 9.5%. Due to the degradation of mechanical properties of the CFRP, the ultimate compressive strength of CFRP confined cylinders was reduced by 12.1%, 10.8% and 13.9% for specimens wrapped with 1, 2 and 3 layers of CFRP respectively. The corresponding reductions were respectively 11.4%, 2.2% and 4.1% for the ultimate axial strain and 16.8%, 5.2% and 15.6% for the ultimate hoop strain.

**Comparison of the experimental results with model predictions**

The test results are compared with the predictions from the ACI 440.2R-08 (2008) design guidelines, and the Chinese (50608-2010 GB) and Italian (CNR-DT 200/2004) standards as shown in Figure 4. The tested rupture strain of CFRP after applying an appropriate environmental reduction factor from these documents was adopted as the design hoop strain. Figure 4 shows that all the three design provisions especially CNR-DT 200/2004 underestimated the impact of the salt solution environment on the ultimate compressive strength when the cylinders were confined with 1 layer of CFRP. When the concrete was confined with 2 or 3 layers of CFRP, the calculated values of all three codes were conservative compared with the experimental results, but the Chinese code (50608-2010 GB) predictions were more conservative than the other two.

**CONCLUSIONS**

This paper has presented an experimental study on the effect of cyclic wet-dry exposure in a 3.5% salt solution on the strength of CFRP wrapped concrete cylinders. With the increase of the number of wet-dry cycles, the degradation of mechanical properties of concrete confined with CFRP under axial compression increases. After 360 wet-dry cycles, the ultimate hoop and axial strain of the confined concrete with one layer of CFRP decreased by 16.8% and 11.4%, respectively. The reduction of hoop strain would directly affect the ultimate compressive strength of the confined concrete, and the decrease of axial strain would reduce the ductility of confined concrete.
When concrete confined with one layer of CFRP, the design provisions in the ACI, Chinese and Italian design guidelines or standards appear to underestimate the impacts of the salt solution environments on the ultimate compressive strength.

**ACKNOWLEDGMENTS**

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TENSILE PROPERTIES OF BASALT-FIBRE REINFORCED POLYMER (BFRP) BARS WITHIN SEAWATER AND SEA SAND CONCRETE ENVIRONMENT

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ABSTRACT

This paper presents an investigation on the tensile properties (including tensile strength and Young’s modulus) of BFRP (basalt-fibre reinforced polymer) bars within SWSSC (seawater and sea sand concrete) solution using the accelerated corrosion tests. Three temperatures (room temperature, 40°C and 55°C) were adopted in the testing program. After exposed in SWSSC solution for 21, 42 and 63 days, respectively, each specimen was tested to failure. The test results showed that the tensile strength degradation rates of BFRP bars within SWSSC solution are slightly higher than those under alkaline solution generated from normal concrete at room temperature and 40°C condition, while at 55°C SWSSC solution appears much more aggressive than alkaline solution. The Young’s modulus of all BFRP bar specimens basically keeps unchanged after exposure to SWSSC solution for 63 days. Finally, the prediction of long-term performance of BFRP bars under SWSSC environment at five Australian capital cities is proposed, and the expected time corresponding to the tensile strength retention 50% was conservatively estimated.

KEYWORDS

BFRP bars, tensile properties, sea water and sea sand concrete (SWSSC), long-term performance.

INTRODUCTION

The degradation of reinforced concrete (RC) structures is mainly caused by the corrosion of steel bars inside structures. To overcome this problem, fibre reinforced polymer (FRP) bars are proposed to replace the traditional steel in civil construction due to their excellent corrosion resistance and high strength-to-weight ratio. Recently, basalt fibre reinforced polymer (BFRP) has been developed because basalt fibre possesses higher mechanical behaviour and better chemical resistance than E-glass fibre and remarkably lower production cost than carbon fibre (Lu et al. 2015). Besides, basalt fibre is an environmentally friendly and non-hazardous material as it is directly produced from volcanic rocks without additives.

The increasing use of BFRP bars in construction field necessitates a comprehensive investigation on the durability of BFRP bars under various civil engineering environments. However, only limited works (Wu et al. 2014; Benmokrane et al. 2015) were reported on the long-term durability of BFRP bars. Wu et al. (2014) systematically investigated the durability of BFRP bars pre-exposed to four different simulated corrosion environments (i.e. concrete pore solution, seawater/deicing salt, acid rain and damp environments). The test results showed that the tensile strength retention of BFRP bars after 42-day pre-exposure to alkaline concrete pore solution at 40°C is 88.9%, which is obviously lower than those of BFRP after exposure for the same duration to deionized water, salt and acid environments at 40°C. It was also proved that the sustained loading can effectively accelerate the degradation of BFRP bars in alkaline solution because compared with the case without any pre-loading the tensile load makes the microcracks in the resin matrix easier to initiate.

Compared with the conventional normal concrete, the seawater and sea sand concrete (SWSSC) has a distinct advantage including saving fresh water and river sand (Li et al. 2016). Conventional steel bars can be replaced by BFRP bars to provide internal reinforcement to SWSSC. Very limited work has been done on the durability of BFRP bars under the SWSSC environment (Dong et al. 2016a). To fill this knowledge gap, the tensile properties
of BFRP bars within SWSSC environment were studied, using the accelerated corrosion tests in the present study. In addition, the long-term behaviour of BFRP bars under simulated SWSSC solution was predicted with the Arrhenius relationship model.

EXPERIMENTAL PROGRAM

Materials

The unidirectional BFRP bars with a normal diameter of 6mm used in this research program were identical to those in Wu et al. (2014). BFRP bars were made of continuous basalt fibres (approximately 65% fibres by volume) impregnated in a Bisphenol A epoxy resin using the pultrusion process. The basalt fibres were manufactured by ZheJiang GBF Basalt Fiber Co. Ltd. (Hengdian, China). The fibre had a tensile strength of 3800-4840 MPa, an elastic modulus of 93.1-110 GPa. The epoxy resin was procured from Bluestar New Chemical Materials Co. Ltd (Wuxi China). The reported tensile strength, elastic modulus and elongation of unconditioned BFRP bars were 1398 ± 30 MPa, 46 ± 0.4 GPa and 3.0 ± 0.07% (Wu et al. 2014), respectively.

Simulated Seawater and Sea Sand Concrete Solution

In practice, the BFRP bars embedded in seawater and sea sand concrete (SWSSC) structures can be exposed to moisture, chloride and alkali conditions. The simulated SWSSC pore solution was prepared by mixing: sodium hydroxide (2.4 g/l, NaOH), potassium hydroxide (19.6g/l, KOH), calcium hydroxide (2 g/l, Ca(OH)_2), sodium chloride (35 g/l, NaCl) and distilled water (to make the volume of solution of one litre). The mass concentrations of NaOH, KOH and Ca(OH)_2 were chosen according to Chen et al. (2007) to simulate the normal concrete pore solution. The measured pH of SWSSC solution is 13.5-13.7.

Test Plan

To evaluate the durability of BFRP bars, the tensile tests of reference specimens and exposed specimens were conducted in accordance with ACI-440.3R B.2. The length of the tensile specimen was 760 mm. Each end was coated with silica sands and then placed into 250 mm-long steel tubes grouted with an epoxy resin mortar matrix. Aluminium caps were used to close both ends of the steel tubes and to keep the bar in the centre of the pipe during anchor installation. The reference specimen thus prepared is shown in Figure 1(a). The length of the test section was 260 mm. The test was carried out by gripping the steel tube into the wedges of a Baldwin machine with a capacity of 500 kN. A loading rate of 2 mm/min was adopted and string potentiometers were used to measure the deformation of BFRP bars.

As shown in Figure 1(b), a solution circulating system was designed for the exposure experiment. The middle part of the tensile specimen was inserted inside a 30 mm-diameter and 240 mm-long PVC pipe worked as the container for exposure to the SWSSC solution. Both ends of the PVC pipe were sealed with epoxy adhesive to ensure a perfect closure during conditioning. The SWSSC solution was contained in a beaker with 5 L capacity that was placed in the thermostatic water bath to maintain the temperature. A submerged pump was used to circulate/transport the SWSSC solution through the whole circulating system.

Figure 1 Images of BFRP tensile specimens
Three exposure temperatures (room temperature, 40 and 55 °C) were adopted for the simulated environment. After exposure in SWSSC solution for 21, 42 and 63 days at each temperature, BFRP specimens were removed from the aging system and tested under tension to compare their tensile strength with that of the reference specimen. A summary of the test matrix is presented in Table 1. All the pre-exposure experiments were conducted in the indoor environment. The room temperature of SWSSC solution at room temperature condition during the whole exposure period is monitored as 32.2 ± 1.2°C. Therefore, the “room temperature” in this study refers to 32°C.

### Table 1 Test matrix

<table>
<thead>
<tr>
<th>Aging condition</th>
<th>Temperature (°C)</th>
<th>Aging time (days)</th>
</tr>
</thead>
</table>
| Reference       | N/A              | 3
| SWSSC solution  | 32°              | 2
|                 | 40               | 2
|                 | 55               | 2

Note: a: room temperature; b: number of tensile specimens used for each group; N/A: not available.

### RESULTS AND DISCUSSIONS

#### Tensile Test Results

**Tensile strength retention**

The tested tensile strength of reference BFRP specimens was 1351 ± 74 MPa, which was close to the reported data (1398 ± 30 MPa) in Wu et al. (2014). Corrosion damages were caused by SWSSC and alkaline solution (the later represents normal concrete pore solution) to the BFRP bar. The tensile retention results of BFRP specimens pre-exposed to the two different solutions at different temperatures were compared (Figure 2). The solid lines represent the data for the SWSSC solution, which is compared with the data for the alkaline solution that were extracted from Wu et al. (2014). This shows that the tensile strength of BFRP bars decreases with an increase in exposure period for both solutions at all temperatures, and the degradation was more severe at higher temperatures.

After 63 days of exposure to SWSSC solution at 32 and 40 and 55°C, the tensile strength retention was 92.7%, 81.7% and 26.0%, respectively. After 63 days of exposure to alkaline solution at 25, 40 and 55°C, the tensile strength retention is 94.0% and 84.8% and 68.5%. These data suggest that SWSSC solution has caused more damage to BFRP than alkaline solution, especially at the higher test temperature, 55°C. This phenomenon is mainly due to that SWSSC solution has obviously higher concentration of alkali-ions than alkaline solution that is used in Wu et al. (2014). It was reported (Coricciati et al. 2009) that the crystallographic structure of basalt is made by olivine (single tetrahedron), pyroxenes (linear chain) and plagioclase (tetrahedral space structure). The great reduction in strength is attributed to the breaking of some bonds of the linear tetrahedral chain of pyroxenes during exposure to the aggressive alkaline environment (Coricciati et al. 2009).

![Figure 2 Comparison of tensile strength retention of BFRP in SWSSC and alkaline environments at different temperatures](image)

**Young’s Modulus**

As shown in Figure 3, all the stress-strain relationship curves under different exposure conditions show a lineally elastic behaviour until tensile failure. Failures were typically of brittle nature for FRP bars with unidirectional fibres. In addition, the linear regions of all curves are very close to each other, suggesting a negligible change in their Young’s modulus. This result also indicates that the Young’s modulus of BFRP bars is not affected by
exposure to SWSSC solution. Similar test results for BFRP bars and GFRP bars under various environment conditions were also reported in reference (Robert and Benmokrane 2013; Wu et al. 2014). This is possibly explained by the fact that the modulus of unidirectional FRP bars mainly depends on the modulus of basalt fibre, while the degradation of modulus of basalt fibre under SWSSC solution may not be significant.

Figure 3 Change of stress-strain relationship curves of BFRP exposed to SWSSC solution at different temperatures for various durations

Surface appearance

Figure 4 shows the surface appearance of BFRP specimens that were pre-exposed to SWSSC solution for 63 days at different temperatures. As shown in Figure 4, the surface colour of all exposure specimens changed obviously compared with the reference specimen. Moreover, the severe surface fuzz and porousness occurred clearly for the specimen at 55°C condition. It is believed that the degradation of matrix and fibre-resin interphase were both significant, which was mainly caused by the SWSSC solution attack at high temperature. The surface appearance is also consistent with the tensile strength results.

Figure 4 Surface appearance of BFRP specimens after 63 days exposition to SWSSC solution at different temperatures: (a) reference; (b) 32°C; (c) 40°C and (d) 55°C

PREDICTION OF LONG-TERM BEHAVIOUR

Arrhenius Relationship

The degrade rate can be expressed in the Arrhenius relationship by the following equation (Wu et al. 2014):

\[
k = A \exp\left(-\frac{E_a}{RT}\right)
\]

where \(k\) = degradation rate (1/time); \(A\) = constant of the material and degradation process; \(E_a\) = activation energy; \(R\) = universal gas constant; and \(T\) = Kelvin temperature. The basic assumption of this model is that the single dominant degradation mechanism of the material does not change with time and temperature during the exposure, whereas the rate of degradation is accelerated with the increase in temperature. Eq. (1) can be transformed into:

\[
\frac{1}{k} = \frac{1}{A} \exp\left(\frac{E_a}{RT}\right)
\]

\[
\ln\left(\frac{1}{k}\right) = \frac{E_a}{RT} - \ln A
\]

Eq. (2) shows the degradation rate \(k\) can be expressed as the inverse of the time needed for a material property to reach a given value. Eq. (3) shows the logarithm of time needed for a material property to reach a given value is a linear function of \(1/T\) with the slope of \(E_a/R\) value.
Prediction Procedure

Firstly, the following degradation model was used to define the relationship between tensile strength retention of BFRP bars and exposure time for the accelerated test:

\[ Y = 100 \exp (-t/\tau) \]  

where \( Y \) = tensile strength retention (%); \( t \) = exposure time; and \( \tau \) = fitted parameter.

On the basis of Eq. (4), the fitted curves are shown in Figure 5 and corresponding \( \tau \) values and correlation coefficients (\( R^2 \)) are listed in Table 2. The \( R^2 \) of all the curves are above 0.88.

\[ \text{Figure 5 Fitted curves for tensile strength retention versus time} \]

Table 2 Coefficients of the regression equations in Eq. (4)

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>BFRP bars in SWSSC solution</th>
<th>( \tau )</th>
<th>( R^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>32</td>
<td>719.2</td>
<td></td>
<td>0.88</td>
</tr>
<tr>
<td>40</td>
<td>325.8</td>
<td></td>
<td>0.99</td>
</tr>
<tr>
<td>55</td>
<td>46.5</td>
<td></td>
<td>0.99</td>
</tr>
</tbody>
</table>

Secondly, with the regression coefficient \( \tau \) listed in Table 2 the time (\( t \)) needed for the tensile strength retention to reach 60, 70, 80, and 90% at 32, 40, and 55° C were calculated from Eq. (4). Next, Eq. (3) was fitted to those values (\( \ln(1/k) = \ln t \) and \( 1/T \)), and the straight lines were exactly parallel. The slopes of the straight lines \( (E_a/R) \) were 12082 and the correlation coefficients (\( R^2 \)) are all over 0.98. Due to the limited space, the details of fitted curves are not given in this manuscript. Thirdly, based on Eq. (1), the time-shift factor (TSF) for the tensile strength to reach the same value (represented by \( c \)) at temperatures \( T_1 \) and \( T_0 \) can be obtained from the previous Arrhenius plots. The TSF can be expressed as:

\[ \text{TSF} = \frac{t_0}{t_1} = \frac{c/k_0}{c/k_1} = \frac{k_1}{k_0} = \frac{\exp(-E_a/RT_1)}{\exp(-E_a/RT_0)} = \exp \left[ \frac{E_a}{R} \left( \frac{1}{T_0} - \frac{1}{T_1} \right) \right] \]  

In this study, five capital cities located in Australia were chosen to predict the long-term behavior of BFRP. The annual mean temperature of each city is listed in Table 3. According to Eq. (5), the TSF with reference temperatures \( T_0 \) equal to annual mean temperature of each chosen city are calculated and listed in Table 3.

Table 3 Time-shift factor

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>Darwin (^{27.6°C})</th>
<th>Brisbane (^{21.4°C})</th>
<th>Sydney (^{17.8°C})</th>
<th>Melbourne (^{15.1°C})</th>
<th>Hobart (^{12.7°C})</th>
</tr>
</thead>
<tbody>
<tr>
<td>55</td>
<td>28.6</td>
<td>66.7</td>
<td>110.8</td>
<td>163.4</td>
<td>232.4</td>
</tr>
<tr>
<td>40</td>
<td>4.9</td>
<td>11.4</td>
<td>19.0</td>
<td>28.0</td>
<td>39.8</td>
</tr>
<tr>
<td>32</td>
<td>1.8</td>
<td>4.2</td>
<td>6.9</td>
<td>10.2</td>
<td>14.5</td>
</tr>
</tbody>
</table>

Note: a: annual mean temperature, the average of Annual Mean Maximum Temperature and Annual Mean Maximum Temperature from Australian Government Boricua Meteorology online database. (http://www.bom.gov.au/climate/averages/tables/ca_site_file_names.shtml)

Once the TSF values for 55, 40 and 32 °C were obtained, Figure 5 was transformed into Figure 6 by multiplying exposure times at 55, 40 and 32 °C with corresponding TSF values. Finally, master curves for tensile strength retention versus exposure time at five different cities were obtained by fitting Eq. (4) in Figure 6. From the master curves, the tensile strength retention of BFRP bars in SWSSC solution drops to 50 % after 2.6 years, 6.0 years, 10.0 years, 14.8 years, and 21.0 years at Darwin (27.6°C), Brisbane (21.4°C), Sydney (17.8°C), Melbourne
(15.1°C), and Hobart (12.7°C), respectively. It should be pointed out that this estimation may be conservative. On the one hand, the humidity of the concrete in service condition is obviously lower than that of simulated solution (Dong et al. 2016b). On the other hand, the temperature inside the concrete may be much less than the environmental temperature. More research is needed to clarify this issue.

Figure 6 Master curves of BFRP bars exposed to different annual temperatures in five Australia capital cities

CONCLUSIONS

(1) At room temperature and 40°C, SWSSC solution caused slightly more damage to BFRP bars when compared with the damage caused by alkaline solution generated from normal concrete (Wu et al. 2014). When the aging temperature reaches to 55°C, SWSSC solution was found more severe than concrete-pore alkaline solution.

(2) Nearly no change was found in Young’s Modulus of BFRP bars after exposure in SWSSC solution for 63 days. This is mainly because the modulus of BFRP bars depends on the Young’s Modulus of basalt fibre, while the degradation of modulus of basalt fibre under SWSSC solution may not be significant.

(3) Arrhenius relationship theory was adopted to predict the long-term performance of BFRP bars under SWSSC solution at five capital cities in Australia.

ACKNOWLEDGMENTS

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AN EXPERIMENTAL STUDY ON THE THERMOMECHANICAL AND RESIDUAL BEHAVIOUR OF THE PULTRUDED CARBON FIBER REINFORCED POLYMER LAMINATE SUBJECTED TO HIGH TEMPERATURE LOADING

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ABSTRACT

In civil engineering, when pultruded carbon fiber reinforced polymer (P-CFRP) laminate-reinforced structures are subjected to fire, the P-CFRP material is simultaneously affected by high temperatures (potentially up to 1200°C) and mechanical loadings. This paper presents an experimental comparison between the thermo-mechanical behaviour and the residual one of P-CFRP material subjected to the temperature levels ranging from 20°C to 600°C. The evolution of the ultimate tensile stress and of the elastic modulus of the pultruded CFRP material as a function of the temperature level has been experimentally identified. The results show a gradual decrease in ultimate stress and in elastic modulus of P-CFRP material as the temperature level rises and the rate of this reduction increases with the rise of this level. The P-CFRP material ultimate stresses obtained from the thermo-mechanical tests are always lower than those obtained from the residual tests for the same temperature level (in particular for temperature levels above 400°C). Additionally, the P-CFRP material failure modes as a function of the loading paths at different temperature levels are also analysed and discussed.

KEYWORDS

Carbon fibre reinforced polymer (CFRP), high temperature, thermo-mechanical behaviour, residual behaviour.

INTRODUCTION

Pultruded carbon fibre reinforced polymer (P-CFRP), a polymer-matrix composite reinforced by carbon fibre, is commonly used in industries such as aerospace, automotive, civil engineering for its high strength-weight ratio, good properties in corrosion resistance and fatigue. In construction industry, CFRP is applied to repair structures or to increase the strength and ductility of concrete structures such as columns beam and slab. When the CFRP reinforcing structures is subjected to fire, the CFRP material is exposed to very high temperature up to 1200°C while undertaking mechanical loading (service load). In the literature, there are few experimental researches which study the residual behaviour of fibre reinforced polymer (FRP) subjected to fire (Adelzadeh et al, 2014 ; Feih et al, 2012; Correia et al, 2010; Foster et al, 2005). There are also much fewer experimental studies regarding the thermo-mechanical behaviour of CFRP at high temperature (Y. C. Wang et al, 2007; K. Wang et al, 2011). Base on discrete testing results from different type of material, there is a proposal evolution for the properties of CFRP at high temperature (Saafi, 2002). As far as the knowledge of author, there is not available comparative study concerning the thermo-mechanical and residual behaviour of CFRP material subjected to high temperature condition. This research is aimed to clarify the difference between two working cases of pultruded CFRP material as well as to develop material property data concerning the variation of strength and stiffness of CFRP at different levels of temperature.

EXPERIMENTAL METHODS

Experimental device

In this research, the used device, called TM20kN-1200C, is a system including three major parts: mechanical part, furnace and laser sensor (Figure 1). The mechanical part (Figure 1a) is a direct tensile machine which can provide axial tensile force up to 20 KN. The furnace (Figure 1b, Figures2) is an oven which can generate high temperature up to 1200°C with the maximum heating rate of 30°C/minute. The third part, the laser sensor (c), includes two laser diodes and two cameras, allows measuring the displacement of the concerned points. When the tested material
is homogeneous as pultruded CFRP, the measurement of the laser sensor can provide the axial strain of the part of the specimen placed in the furnace during the test. During this, via laser sensor, the strain of the material will be observed and recorded. The laser sensor of this machine, placed outside the furnace, is regularly calibrated to ensure their reliability. The development and validation of the laser sensor measurement have been discussed in previous works (T.H. Nguyen et al, 2016). The previous studies show that the laser sensor measurement of the used machine is reliable. This research uses the testing system to study the behaviour of pultruded CFRP material under the effects of thermal and mechanical load. This is very important because when a fire happens, structures like beams, columns, slabs and all the reinforced materials are subjected to the heat and supposed mechanical loads at the same time. The testing system is programmed to control the furnace and the mechanic part at the same time so that the mechanical loading and the heating will act on the sample. All the data including force, temperature and strain will then be collected and analysed to study the actual behaviour of material/structure under different loading paths.

**Experimental specimens**

In this research, the studied CFRP (Sika® CarboDur® S512) is pultruded–carbon-fibre plate and contains 68% of carbon fiber. As in the product’s specification, its tensile strength is 2800 MPa, its tensile modulus is 165GPa. Glass transition temperature, Tg is above 100°C. The CFRP product is provided in the standard rolls of laminate with the dimension of 1.2mm in thickness, 50mm in width and 25m in length. Due to the dimension, capacity of testing system, the CFRP is cut in to dimension: 800mm in length and 25mm in width. In order to effectively transmit the tensile force from the mechanic parts to the CFRP material, two aluminium plates is attached in each end by epoxy (two components with commercial name of Etancol 492). As the product specifications of Etancol 492, its shear resistance, tensile resistance and compression resistance are 15 MPa, 29.5MPa and 83 MPa respectively. However, the smooth surface of CFRP laminate leads to low bond strength of the adhesives. This results in an unexpected mode of failure that is occurred at the anchorage position. There are many types of anchorage systems that have been tried by a large number of studies. There are two reliable methods that are appropriated to be applied on the CFRP specimen. They are expansive cement confined by a circular steel tube that is suitable with CFRP tendon (Y. C. Wang et al, 2007), and the aluminium plates embraced with bolts that is suitable to for specimen in thin plate form. The later form is effectively applied to thin plate textile reinforced concrete (R. Contamine et al, 2011). Therefore, the aluminium plates are required to be embraced to the CFRP to ensure that the failure will occur in the test specimen, not in the anchorage region. In this research, based on the testing capacity of the system, the properties of the adhesive and CFRP material, there are two methods to embrace the aluminium plates, with bolts (Figure 3a) and with clamp (Figure 3b). The first is intentionally applied to samples that are tested at low temperature condition while the latter is expected to work at high temperature testing condition. As the instruction from the producer of Etancol 492, the adhesives will reach 83% of its tensile strength after 3 days that is applied onto the joint. In this research, the minimum duration for this waiting time is 7 days (Figure 4a,b). The samples then are trimmed down as the initial configuration (Figure 4c). The specimen’s width in this research varies from 6mm to 8mm due to the constraints from the testing capacity of the machine and tensile resistance of CFRP laminate.

**Experimental procedure**

In this study, there are used two loading paths: thermomechanical test (TM) and residual resistance one (RR). In the thermomechanical tests, Regime 1 (Figure 5), the temperature rises to desire one ($T_{target}$). Once the furnace temperature reaches the target value, it is then kept constant for one desired exposure duration ($T_w$). The force, applied to the specimen, increases monotonically until the maximum one that the specimen can be resisted ($F_r$).
In this study, direct tensile tests, at different target temperature levels (20°C, 200°C, 400°C, 500°C, 600°C, 700°C) and with a thermal exposure duration of one hour (T_w=1 hour), have been carried on CFRP specimens. There are two or three exploitable tests for temperature levels (20°C, 200°C, 400°C, 600°C) and one exploitable test for temperature levels (500°C, 700°C) (see Tables 3 and 4). The thermal exposure duration (T_w=1 hour) is chosen so that the temperature within the tested material is uniform and the measurement of the axial strain by the laser sensor can be performed (T.H. Nguyen et al, 2016). Indeed, during the raise in temperature of 20°C to the desired temperature level T and the beginning of the temperature exposure duration, there are fumes emitted from pultruded CFRP, that disturb the measurement of material axial strain. The laser sensor is so activated after that the studied material specimen is exposed to a desired temperature during 1 hour and the beginning of the raise of the mechanical loading (see Figure 5). In the residual resistance tests, Regime 2 (Figure 6), CFRP specimens are preheated to the same target temperature levels (T_target) and with the same exposure duration (T_w=1 hour) (the same as the thermomechanical tests). They are then naturally cooled in the furnace until reach the state at ambient temperature. The quasi-static and monotonous mechanical load is then applied to these "cooled" specimens to failure (F_r). With testing regime 1, during the duration that the specimen is exposed to high temperature, the middle part (observing region, Figures 3a,b) of sample is heated while two ends are not. During this time, the condition of observing region and two end-regions are working in different states. During the heating phase and the exposure duration, a small initial tensile force (100 N) is applied on specimen and controlled stable until the end of the exposure duration phase. Thanks to this technical solution, both extremities of the specimens are clamped (restrained) during the heating phase and the exposure duration.

In order to investigate the thermo-mechanical behaviour of CFRP material, it is necessary to measure the strain development of CFRP specimen only in the observing region. However, during high temperature testing condition (700°C, in this research), it is difficult to measure the strain in this position for the insufficient of testing device. In this research, the heating room is closed and isolated. A small rectangular glass window is designed on the side of the heating room, enough for the laser rays to pass through and measure the strain of the CFRP in the observing region (Figure 3a). This allows maintaining the stable and homogeneous temperature inside the heating room even at very high temperature testing condition (up to 1200°C). This system objectively measures the deformation of CFRP in the observing region without directly interact with the sample, which reduces the possibility of distortion.
to the deformation data. For each test, the temperature inside the heating room is continuously measured via three thermocouples at top, middle and bottom positions of the observing region (Figure 3b, Figure 2). The testing program bases on these three temperature data to control the level and the homogeneity of the temperature inside the heating room. During the time the mechanical load is applied, the temperature surrounding the sample is maintained stable and homogenous, same as the level of testing temperature with the regime 1, and at the normal-room temperature with the regime 2.

RESULTS

Figures 7 and 8 present the stress-strain curves obtained with the tests performed on the pultruded CFRP according to the thermomechanical regime (Figures 7) and the residual resistance regime (Figure 8). In these figures, there is one representative “stress/strain” curve for each temperature level. Each representative “stress/strain” curve is chosen among the curves obtained with the tests carried out in the same condition and on different specimens of the pultruded CFRP. The test is terminated when the specimen is loaded to failure. In all test, as intended, the fracture happens in the observing region in the specimen, not in the anchorage with the aluminium plates (Table 1, Table 2, Table 3). The type of failure is various depending on the level of temperature that the specimen is exposed to (Table 2, Table 3). This is because that the polymer matrix is softened between 300°C-400°C, and partly burns at 600°C. At 700°C, the matrix is burnt out after 17 minutes exposing to this temperature. In regime 1, the laser performs reliably in measuring the strain of the specimen even in very high temperature condition (up to 600°C). However, at higher temperature, 700°C, when all the polymer matrix all burns out, there is no flat surface left. This leads the laser device fail to capture the data of deformation.

<table>
<thead>
<tr>
<th>Sample’s ID</th>
<th>T°C</th>
<th>Ultimate stress (MPa)</th>
<th>Young modulus (GPa)</th>
<th>Sample after test</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1-20-a</td>
<td>20</td>
<td>2051</td>
<td>147.6</td>
<td></td>
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<td>C1-20-b</td>
<td>20</td>
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<td></td>
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<tr>
<td>C1-20-c</td>
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<td>172.3</td>
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<table>
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<th>Ultimate stress (MPa)</th>
<th>Young modulus (GPa)</th>
<th>Sample after test</th>
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<tbody>
<tr>
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<td>1642</td>
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</tr>
<tr>
<td>C1-TM-200-b</td>
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<tr>
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<th>Young modulus (GPa)</th>
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<td>C1-RR-400-c</td>
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<td>C1-RR-600-b</td>
<td>600</td>
<td>398</td>
<td>54.2</td>
<td></td>
</tr>
</tbody>
</table>

DISCUSSION

For both regimes, it is clear that the ultimate strength of CFRP material is reduced as long as the temperature level is increased. For results obtained from regime 2 (Figure 9), firstly, it is shown that when the temperature level increases from 20°C to 200°C, the ultimate strength decreases significantly. Between 200°C and 400°C, the
ultimate strength reduces more gradually. From 400°C until 600°C, the ultimate strength decreases steadily. For the thermo-mechanical test, the result of ultimate strength shows the similar reduction trend. These results confirm the general reduction trend that is similar to experimental results obtained in the literature (Y.C.Wang et al, 2007; K.Wang et al, 2011); it is significantly different from the prediction model which shows a linear reduction when the temperature increases from 100°C to 400°C (Saafi, 2002) (Figure 10). Secondly, the results from two regimes also show a reduction in elastic tensile modulus of CFRP when the temperature increases (Figure 11). From 20°C to 200°C, the elastic modulus increases less than 1%. When the temperature increases up to 400°C, this modulus decreases about 10% to that at 20°C. However, at the temperature level from 400°C to 600°C, the elastic modulus decreases significantly from 15% to 70% with regime 2 and 85% with regime 1. There is a small different between these result and experimental results in the literature review (Y.C.Wang et al, 2007), but the difference is significant to prediction model proposed by (Saafi, 2002) (Figure 12). The results from regime 1 show that the mode of failure is depended on the temperature range that CFRP is exposed (Table 2). At 20°C, the CFRP is broken at randomly position in the middle of specimen (failure mode A). With the temperature between 200°C and 400°C, the specimen failure happens in the middle of specimen, however, the plate of CFRP is still able to keep its normal shape (failure mode B). From 500°C to 600°C, the specimen failure happens in the middle of specimens, but the CFRP plate is unable to maintain its normal shape (failure mode C). At temperature that above 700°C, the CFRP is broken due to the burning of all material (failure mode D). These results confirm what have been observed by (K. Wang et al, 2011).
In this study, the tests with regime 2 only show similar in evolution trend of failure mode, however, at 400°C, CFRP has the tendency to failure following mode C (Table 3). In this research, the results show that the value of ultimate strength is always lower than that obtained from residual test at each temperature level (Figure 9). At the temperature from 20°C to 200°C, the ratio of ultimate strength obtained from regime 2 to that of regime 1 is small. However, when the temperature increases to 400°C, the ratio becomes greater and this value is more significant at 600°C (Figure 13). Specifically, when the temperature level increases between 20°C-400°C, the ratio of residual to thermo-mechanical performance is low, it increases from 1 to 1.27. When temperature level increases from 400°C to 600°C, this ratio steadily increases from 1.27 to 4.98. In addition, the elastic modulus obtained from regime 1 is also lower than that of regime 2 (Figure 11). The ratio of elastic modulus obtained from regime 2 to that of regime 1 is small at the temperature below 500°C but becomes greater at higher temperature level than that (Figure 14). Particularly, the ratio of elastic modulus obtained from regime 2 to that from regime 1 is about 1 at the temperature from 20°C to 500°C, but from 500°C to 600°C, it increases from 1.04 to 2.92.

CONCLUSIONS

The aim of this work is to experimentally compare the thermomechanical behaviour and the residual resistance of a pultruded carbon fiber reinforced polymer (CFRP) laminate. The evolution of the ultimate tensile stress and of the elastic modulus of the pultruded CFRP material as a function of the temperature level has been experimentally identified. For the results obtained from the tests, it is clear that the performance of the pultruded CFRP material reduces when the temperature, to which it is exposed, increases. This confirms what have been observed before (Wang et al, 2007; Saafi, 2002). The performance of the pultruded CFRP material during exposing to an elevated temperature levels (in particular for temperature levels above 400°C) is lower than that after cooling to normal temperature. In addition, the failure mode of CFRP material varies depending on temperature level that the material is exposed and the regime that the test is taken. When the temperature level increases, the polymer matrix is burnt with the increase in the ratio of burning part. Therefore, the original carbon fibre loses its skeleton and the failure changes from brittle fibre material to more soften one. In the design of structures incorporating pultruded CFRP strips, the residual properties of CFRP are used as a referent value to estimate the performance of structures in fire. However, the results of this research show that using the thermo-mechanical properties in evaluating performance of the pultruded CFRP material subjected to elevated temperatures will improve the reliability and safety of structures incorporating this material.

ACKNOWLEDGEMENTS

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NUMERICAL ANALYSIS OF THE POST-FIRE STRENGTH OF FRP REINFORCED BRIDGE DECKS

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ABSTRACT
This paper presents an approach for assessing the remaining strength after fire of bridge decks reinforced with fibre reinforced polymer (FRP) bars. In fire incidents not resulting in bridge collapse, the challenge is to evaluate the remaining strength of the bridge to decide on further use. The approach involves estimation of the temperatures in bridge decks under two potential fire scenarios for bridges. The first scenario considers a hydrocarbon fire as a worse-case scenario underneath the bridge while the other examines a fuel spill burning on the top of the bridge deck. The study considers typical slab-on-girder bridges with either steel or concrete girders supporting an FRP reinforced concrete slab. The thermal model is verified against fire tests that have been performed by the authors on FRP reinforced concrete slabs. Material tests on GFRP bars were performed earlier by the authors and provide a data source that can be used with the numerical thermal analysis shown in this paper to assess the remaining strength of FRP reinforced concrete bridge decks.

KEYWORDS
Fire resistance, FRP bars, bridge decks, finite element analysis, thermal analysis.

INTRODUCTION
Concerns about the durability of coated steel reinforcement and the high cost of stainless steel have increased applications of glass fibre reinforced polymer (GFRP) bars in construction. During the service life of a bridge, fire is one the most severe hazards. For example, between 1980 and 2012, 30 highway bridges collapsed due to fire in the United States (Lee et al. 2013). In the same time period, 20 bridges failed due to earthquakes (Lee et al. 2013). In recent years, rapid development of ground transportation and increasing transport of hazardous materials have raised concerns about significant losses in a fire incident. Fire has become a real concern for bridges where they are a significant component of urban infrastructure (Garlock et al. 2012). Fire may be a more critical incident to consider for bridge components reinforced with GFRP bars since such bars are susceptible to high temperatures.

MATERIAL TESTS
In a recent study, the tensile strength at high temperature of two different GFRP reinforcing bars was investigated (Hajiloo et al. 2015). In addition, the bond degradation of GFRP reinforcing bars at high temperature was studied using standard pullout tests (Hajiloo and Green 2015). While the GFRP reinforcing bars lost a considerable amount of bond strength at elevated temperatures, results of pullout tests on specimens heated and then cooled down to room temperature before loading showed that the bond strength was to some extent recoverable. Figure 1(a) shows the results of residual bond strength tests on a widely used GFRP reinforcing bar in Canada. The rate of increasing temperature was set at 5 °C/min based upon the expected rate of heat increase of FRP bars embedded in concrete. To ensure uniformity in heating, samples were held at the target temperature for 15 minutes. Then, the furnace was turned off and the samples were left to cool down to room temperature before they were loaded to failure. To include the effect of sustained loads during heat exposure, some samples were loaded to the expected service load during the heat exposure. The specimens shown with triangle symbols in Figure 1(a) were subjected to a sustained load of 10 kN during the exposure to the elevated temperatures. The specimens shown with diamond symbols in Figure 1(a) were not loaded during the heat exposure. For the specimens without sustained load, the temperatures were in the range of 115-274 °C at the interface between the FRP bar and concrete. Since these bars were not loaded during the heat exposure, no mechanical damage was observed after heating and the specimens were tested.
after 24 hours to determine the recoverable bond strength. For the specimens with a sustained load of 10 kN, the temperature was limited to 140 °C because the specimens failed when the temperature reached 140 °C during heating exposure. For exposures up to 140 °C, the residual bond strength was at least 60% of the original bond strength.

To assess the remaining tensile strength of the FRP reinforcing bars after exposure to fire, the same type of GFRP bar, which was used in the pullout tests, was tested in tension using a similar heating and sustained loading procedure to that used for the pullout specimens. For the tensile tests, each reinforcing bar was loaded to 25% of its room temperature strength, and then exposed to the specific level of temperature. The recovered strengths of the reinforcing bars for the examined temperatures were all above 50% of their strength at room temperature as shown in Figure 1(b). The remaining tensile strength should be sufficient to resist service loads since most design provisions limit the stress in GFRP bars at service conditions to approximately 25% of their ultimate strength.

Figure 1 Remaining (a) bond strength and (b) tensile strength after exposure to elevated temperature

**NUMERICAL MODELLING**

The fire resistance of reinforced concrete members has generally been assessed with standard fire tests. In this study, a numerical method is employed as an alternative to standard fire tests for evaluating the post-fire remaining resistance of structural members. However, fire test results provide a basis for verification of the numerical modelling (Hajiloo et al. 2016). Numerical modelling of heat transfer facilitates prediction of the performance of FRP bars at different locations in the member in either fire exposed or cool zones. Thermal properties of the concrete are significant factors in the model because they determine the heat propagation along the length and through the depth of the concrete. The numerical analyses were conducted using the finite element computer program ABAQUS (Hibbitt et al. 2014), which is capable of running coupled and uncoupled thermo-mechanical problems. The thermal properties of concrete are defined in several sources such as Lie (1972) and the Eurocode (2004). These two sources define thermal conductivity and specific heat in a fairly similar way except that the specific heat defined by Lie spikes between 400 to 600 °C due to the assumed presence of quartz. Moisture content and aggregate type affect the specific heat of concrete, but the effects of moisture become insignificant when temperatures are greater than 200 °C. The Eurocode has defined a peak value for the specific heat that occurs between 100 and 200 °C depending on the moisture content. For the modelling in this paper, the thermal properties of the concrete were assumed to follow the recommendations of Lie (1972).

Figure 2(a) shows contours of the heat distribution in a FRP reinforced concrete slab (Hajiloo et al. 2016) that was tested under full service load while exposed to the standard ASTM E119 (2015) fire curve. The results show lower temperatures at the supports than in the centre of the slab. In the modelling, the support regions, 200 mm long at both ends of the slabs, were not exposed to heat. Figure 2(b) shows the reliability of the model by comparing the numerical results with experimental measurements (Hajiloo et al. 2016) at several locations throughout the depth of the slab at midspan.
OVERHANGING CANTILEVER PORTIONS OF DECKS

One of the critical locations during a bridge fire is the overhanging cantilever portion of decks reinforced with negative moment GFRP reinforcing bars that are particularly vulnerable to a fire on the surface of the bridge.

Post-Fire Performance for a Fire on Top of the Deck (Pool Fire)

For a case of fire on top of the deck, the overhanging length and half of the interior bay (in total 2.6 m) was considered to be exposed to a pool fire from spilled diesel fuel that would burn for three hours while the vehicle was on the deck. Hayasaka (1997) measured the liquid and pool temperatures for an uncontrolled fuel depth. It was shown that, after ignition, the liquid fuel and the pool heated up, and boiling in the liquid fuel was observed. As a result, the liquid pool temperature remained constant until it suddenly dropped as the fuel burnt out. It was shown that, throughout the boiling phase, the temperature of kerosene remained below 300 °C. In this study, the maximum temperature on the surface of the bridge deck was assumed 400 °C since the fuel would be diesel rather than kerosene.

The temperatures along the top reinforcing bars, which are placed 50 mm deep from the top slab, are shown in Figure 3. Temperatures dropped dramatically at locations away from the spill area. The temperature along the reinforcing bar in the fire exposed zone reached approximately 160 °C after three hours. According to the steady-state bond strength tests, there is a little bond left at temperatures above 150 °C (Hajiloo and Green 2015). However, as long as the FRP bars have continuity into the cool zone, failure due to bond loss would not occur (Nigro et al. 2011; Hajiloo et al. 2016). In the fire-exposed zone, temperatures of 160 °C would reduce the residual tensile strength of the bars by less than 10 % as shown in Figure 1(b). In the cooler anchor zone, the temperatures were predicted to be below 50 °C (Figure 3) and thus no significant loss of residual bond strength would occur as shown in Figure 1(a). Assuming that the bridge deck survives the fire, based on the study of the pool fire on the deck and indications from the material testing (Figure 1), the remaining post-fire strength of the FRP reinforced concrete deck slab should not be affected significantly by this type of fire as long as the FRP bars have continuity into the cool zone. More detailed structural analyses will be needed to confirm this preliminary assessment.

To conduct such structural analyses, the thermal field developed in the bridge deck can be predicted from the numerical analysis presented in this paper considering different fire scenarios. Then, the amount of damage to the reinforcing bars can be assessed based on the available experimental information on the remaining post-fire strength of FRP bars, and thus the overall remaining strength of the bridge component can be estimated. Structural modelling is currently being conducted by the authors to implement this methodology in more detail. In this approach, different configurations of reinforcing bars (e.g., length of reinforcing bars in the fire-exposed area, the development length of the reinforcing bars, and possible splices in the fire areas) are being considered. The continuity of the reinforcing bars into cool zones over the girders is very significant since fire tests (Nigro et al. 2011; Weber 2008; Hajiloo et al. 2016) revealed that bond deterioration and the length of the anchor zone into cooler parts of the concrete control the fire resistance of FRP reinforced concrete flexural members.
Post-Fire Performance for a Fire from Underneath

For a fire from underneath the bridge, the overhanging length to the centreline of the exterior girder is subjected to heat exposure. In this case, a hydrocarbon (tunnel) fire is considered as a worse-case scenario. Most bridges are in open areas and thus the temperatures would not build up to the same degree as they would in a tunnel. Numerical thermal analyses were performed on the overhanging (cantilever) portion of the bridge deck with a 200 mm thickness and a 1500 mm overhanging length from the centre of the steel girder. The hydrocarbon temperature-time curve (European committee for standardization 2002) used in this study is shown in Figure 4(a). For the overhanging portion of the bridge deck, Figure 4(b) shows that a hydrocarbon fire from underneath of the bridge would not increase the temperature in the top layer (150 mm from the bottom of the slab) of the reinforcing bars to more than 50°C and thus the post-fire strength of the bars would not be reduced.

INTERIOR BAY

Thermal analysis for fire in the interior bay showed that both the type of girders (steel or concrete) and the size of girders play a significant role in providing cool zones. For bridges with steel girders, the steel will be severely damaged when the fire temperature exceeds 600°C, and the bridge will fail because of girder failure prior to any strength deterioration of FRP bars.

Figure 5 shows temperatures in reinforcing bars in an FRP reinforced concrete deck on concrete girders 400 mm wide with exposure to a hydrocarbon fire from underneath. The temperature profiles demonstrate that the concrete girder can play a protective role for the FRP reinforcing bars since the temperatures along the bottom bars drop significantly over the girder area thus creating a cool zone to protect the bond strength of the FRP reinforcing bars.
If the FRP bars are continuously extended into the regions over the girders (i.e., all splices over the girders rather than in the spans), then the FRP reinforcement should maintain at least 50% of its original strength after a fire. As previously mentioned, such strength retention should be sufficient for most designs that are governed by serviceability requirements. However, the exposure to fire may change the properties of the polymer matrix in the FRP and thus the long-term properties of the FRP may be affected. More research is needed to investigate such effects.

![Image of FRP bars in bridge deck](image)

**Figure 5 Temperatures along the top and bottom bars due to hydrocarbon fire (Three-hour exposure)**

For the bridge deck with concrete girders, the parts of the deck located over the girders would need to have a sufficiently long cool zone to provide the required embedment length to transfer the forces in the reinforcing bars to the concrete. More research is required to determine the minimum length of this cool zone to provide sufficient embedment.

For a pool fire on top of the deck, temperatures will rise around the FRP bars while the deck carries the dead load and live load due to a burning vehicle. For the interior spans, the temperature increases in the top FRP reinforcing bars are not critical because the bottom layer carries the forces due to positive moment in the span. For the negative moment area, the forces in the top reinforcing bars due to dead and live loads have to be calculated, and strength of the FRP reinforced concrete deck can be investigated using the information on residual material properties after exposure to elevated temperatures.

**CONCLUDING REMARKS**

Nonlinear finite element analyses were conducted to provide a preliminary assessment of the post-fire remaining strength of FRP reinforced concrete decks. The modelling showed that the maximum temperatures of the negative FRP reinforcing bars remained around 160 °C due to an assumed pool fire extending into the cantilever portion of the bridge deck. Thus most of the original strength of the deck would likely be recoverable based on the results of residual bond and tensile strength material tests. However, this conclusion was drawn assuming that the pool fire does not spread over the entire deck, and that there is sufficient unaffected length of the FRP reinforcing bars to prevent bond failure. For a fire incident under a bridge, the negative moment reinforcement, which is critical for the overhanging portion of the deck, should not be affected because the modelling showed that the temperatures in these bars remained below 50 °C. Additionally, the results indicated that concrete girders could provide some protection to prevent bond failure of the bottom reinforcement in FRP reinforced concrete decks since the temperatures over the girders were reduced to approximately 50 °C. Finally, these preliminary assessments should be regarded as indications of potential performance of bridge decks after fire scenarios. Additional investigations with more thorough structural analyses and potentially additional fire tests are needed to provide more rigorous conclusions.
ACKNOWLEDGEMENTS

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TENSILE PROPERTIES OF GFRP BARS AFTER EXPOSURE TO HIGH TEMPERATURES

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ABSTRACT

This paper reports a set of tensile tests of glass fiber reinforced polymer (GFRP) bars after exposure to high temperatures. The effects of both temperature and duration of exposure were investigated. The test results show that the stress-strain relationship of the GFRP bars remained close to linear elastic within the range of the tested temperatures (up to 270°C) but both the tensile strength and the ultimate fracture strain of the GFRP bars reduced gradually as temperature rose whilst the tensile modulus of elasticity was less affected. At 270°C, the vast majority of thermal decomposition was completed within half an hour. The bars had little mechanical strength and stiffness left when the exposure temperature was 350°C. A tensile strength reduction factor for the GFRP bars was proposed for high temperatures up to within 270°C.

KEYWORDS

Glass fibre reinforced polymer (GFRP) bars, high temperatures, tensile properties, strength reduction factor.

INTRODUCTION

One of the main causes for deterioration of reinforced concrete structures is the corrosion of steel reinforcement bars. FRP bars have many advantages, such as high tensile strength, light weight, excellent corrosion resistance and durability, electromagnetic resistance etc. (Guo 2006). FRP bars may be used in concrete structures either replacing steel bars or as supplementary reinforcements and have been used in many practical applications. There are a wide range of commercial FRP bars available, including GFRP, CFRP and BFRP bars. Among them, GFRP bars are the most commonly used in concrete structures because of its low cost.

The matrix in FRP bars is usually a polymer material, its bond performance usually degenerates quickly when exposed to high temperatures. As the temperature resistance of FRP bars is poorer than that of steel bars, FRP bars are mainly used in concrete structures where no fire resistance is required. This may significantly limit the scope of application of FRP bars (Gao et al. 2008a). The performance of FRP bars in concrete structures is thus of great interest when they are exposed to fire.

Wang et al. (2007) studied the mechanical properties of CFRP and GFRP bars at elevated temperatures. The experimental results show that the stress-strain relationships of FRP bars remained almost linear at elevated temperatures until failure. However, there was a gradual reduction in the failure strength of FRP bars at elevated temperatures. This reduction is almost linear with temperature and the strength reaches zero at about 500°C. Their elastic modulus remained almost unchanged until 300~400°C. As temperature further increases, there was a sharp drop in the elastic modulus. A temperature of about 350°C appears to be critical for the two FRP bars. Correia et al. (2013) performed an extensive study about the tensile, shear and compressive responses of pultruded GFRP plate and I sections at temperatures varying from 20°C to 250°C. The test results showed that the mechanical performance of the pultruded GFRP was severely deteriorated at 250°C, particularly when loaded in shear and compression, owing to the low glass transition temperature of the resin. Yu and Kodur (2014) studied the effect of temperature on tensile strength and elastic modulus of CFRP strips and CFRP rods as near-surface mounted (NSM) reinforcement (but tested bare), within a temperature range of 20 to 600°C. The results showed that both the CFRP strips and rods retained most of their initial tensile strength and elastic modulus up to 200°C. However, these properties degraded significantly beyond 300°C due to the decomposition of the resins. The test data were used to
propose empirical relationships between strength and modulus of NSM CFRP reinforcements with temperature. Lu et al. (2015) investigated the effects of elevated temperature on the mechanical properties of basalt fibre roving and BFRP plates. For comparison, E-glass fiber roving and GFRP plates were also tested under the same conditions. It was observed that as the temperature increased from room temperature to 200 °C, the tensile strength and the modulus of elasticity were reduced by 37.5% and 31% for the BFRP plates, and by 8.3% and 9.7% for the basalt fibre roving, respectively, due to the deterioration of bond between basalt fibres and the epoxy resin matrix at elevated temperatures. After exposure to 120 °C and 200 °C for 4 hours, negligible degradation on the mechanical properties occurred in the BFRP plates, while a reduction in the Weibull shape parameter (m) was found for the basalt fibre roving. Compared to the glass fiber rovings and GFRP plates, the basalt fibre roving and BFRP plates showed much better tensile properties and temperature resistance.

Although there have been valuable investigations on the mechanical properties of FRP bars after high temperature exposure, relevant test data are still scarce for design and for safety assessment of FRP bar reinforced concrete structures exposed to fire. This paper reports the longitudinal tensile performance of GFRP bars after exposure to high temperatures. Test parameters included the peak temperature and duration of exposure.

**DESIGN OF EXPERIMENTS**

**Materials and parameters**

A total of 50 GFRP bars with a diameter of 8mm were tested in this study. The GFRP bars were produced in Shenzhen. The bar specimens were divided into 10 groups, each with five bars providing sufficient data for measuring the scatter. The specimens were tested in two series: one with varying temperature (laboratory room temperature RT, 70 °C, 120 °C, 170 °C, 220 °C, 270 °C and 350 °C) but a constant exposure time of 6 hours, the other tested at a constant temperature of 270 °C but with exposure time equal to 0.5, 1, 3 and 6 hours.

**Specimen design and test procedure**

The tensile FRP bar specimens were designed and prepared according to the Guide Test Methods for Fiber-Reinforced Polymers (FRPs) for Reinforcing or Strengthening Concrete Structures (ACI440.3R-04). Figure 1 shows the geometry of the specimens. The specimens were exposed to higher temperatures in a GWL series high temperature furnace at a heating rate of 5 °C/min. The temperature was kept constant once it rose to the set value. The temperature fluctuation within the furnace was within ±5 °C. The specimens were allowed to cool down to room temperature once the designed exposure time was reached. They were tested at room temperature following ACI440.3R-04 using a 1000kN hydraulic servo controlled universal testing machine. An extensometer with 100mm gauge length was attached to the test specimen to measure the tensile deformation.

![Figure 1 Geometry of tensile FRP bar specimens](image)

**TEST RESULTS AND ANALYSES**

**Failure modes**

The GFRP bars were originally shining black. As the temperature increased, the shininess was gradually lost. The integrity GFRP bars looked intact with bare eyes when the exposure temperature was below 270 °C, but the resin was carbonised and fell off to expose some red-brown basalt fibres with the GFRP bars distorted when the temperature reached 350 °C.

For specimens without temperature exposure, sounds due to fibre rupture were emitted when the tensile load was increased to about 15% of the tensile capacity. The sound intensity increased with the increase of loading. A specimen usually failed accompanied by a loud noise and very rapid drop of loading. The data logger was set to stop collecting data when the load was dropped to 45% of the peak load. As the exposure temperature increased, the emission of sounds was delayed and the intensity was reduced.
All test specimens failed within the test zone. The failure modes of the GFRP bars under tensile test may be classified into: a) “pitting” failure where outer fibres fractured resulting in a pit (Figure 2a); b) fracture failure of fibres (Figure 2b) and c) fibre splitting failure (Figure 2c). All the specimens with low exposure temperature (including those without temperature exposure and those with exposure temperature up to 220°C) failed due to either pitting or fracture. The fracture surface was white for those without temperature exposure or with very low (70°C and 120°C) temperature exposure, it became toast tan when the exposure temperature was 170°C and 220°C. All specimens exposed to 270°C failed due to fibre splitting during tensile tests with floc like loose mahogany fibres exposed (Figure 2c).

Test results

The test results for tensile strength, elastic modulus and ultimate strain of the GFRP bars are tabulated in Table 1. Each group of specimens was designed by G8 representing the 8mm diameter bars, followed by the exposure temperature with RT standing for room temperature, followed by the number of high temperature exposure time.

In Table 1, P, E, f_u and ε_u represent the ultimate load, tensile modulus, tensile strength and ultimate strain respectively.

<table>
<thead>
<tr>
<th>Designation</th>
<th>P/kN (CoV)</th>
<th>E/GPa (CoV)</th>
<th>f_u/MPa (CoV)</th>
<th>ε_u/% (CoV)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G8-RT</td>
<td>40.9 (7.6%)</td>
<td>54.2 (5.6%)</td>
<td>820 (7.6%)</td>
<td>1.51 (2.3%)</td>
</tr>
<tr>
<td>G8-70°C-6h</td>
<td>40.0 (4.3%)</td>
<td>58.4 (3.0%)</td>
<td>796 (4.4%)</td>
<td>1.36 (6.5%)</td>
</tr>
<tr>
<td>G8-120°C-6h</td>
<td>37.0 (4.6%)</td>
<td>56.8 (3.2%)</td>
<td>736 (4.6%)</td>
<td>1.30 (3.9%)</td>
</tr>
<tr>
<td>G8-170°C-6h</td>
<td>31.6 (5.9%)</td>
<td>55.3 (8.1%)</td>
<td>629 (5.9%)</td>
<td>1.14 (3.3%)</td>
</tr>
<tr>
<td>G8-220°C-6h</td>
<td>31.5 (5.7%)</td>
<td>54.4 (6.0%)</td>
<td>628 (5.6%)</td>
<td>1.15 (4.0%)</td>
</tr>
<tr>
<td>G8-270°C-6h</td>
<td>26.4 (3.4%)</td>
<td>54.1 (4.9%)</td>
<td>526 (3.4%)</td>
<td>0.973 (1.6%)</td>
</tr>
<tr>
<td>G8-270°C-0.5h</td>
<td>31.2 (4.7%)</td>
<td>57.8 (2.0%)</td>
<td>620 (4.7%)</td>
<td>1.07 (6.6%)</td>
</tr>
<tr>
<td>G8-270°C-1h</td>
<td>29.5 (2.6%)</td>
<td>58.3 (2.7%)</td>
<td>586 (2.6%)</td>
<td>1.00 (2.1%)</td>
</tr>
<tr>
<td>G8-270°C-3h</td>
<td>28.7 (9.1%)</td>
<td>56.2 (2.3%)</td>
<td>570 (9.1%)</td>
<td>0.996 (10.4%)</td>
</tr>
<tr>
<td>G8-350°C-6h</td>
<td>9.50</td>
<td>—</td>
<td>190</td>
<td>—</td>
</tr>
</tbody>
</table>
Analysis of test results

Effect of exposure temperature

Table 2 shows the reduction of tensile strength of the GFRP bar specimens after exposure to different high temperatures. When the temperature increased from room temperature to about 200°C exposure temperature, the tensile strength of the GFRP specimens reduced gradually at a low rate, with the GFRP bar specimens retained about 90% of its original strength at 120°C. When the exposure temperature increased further, the GFRP bar specimens experienced much faster strength degradation. At 350 °C, the tensile strength was reduced sharply by about 77%. Note that strength reduction = (tensile strength at room temperature - tensile strength after high temperature exposure) / tensile strength after high temperature exposure) × 100%. Similar calculation applies to the reduction of modulus of elasticity and rupture strain.

<table>
<thead>
<tr>
<th>Exposure temperature</th>
<th>RT</th>
<th>70°C</th>
<th>120°C</th>
<th>170°C</th>
<th>220°C</th>
<th>270°C</th>
<th>350°C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reduction of tensile strength, %</td>
<td>NA</td>
<td>2.91</td>
<td>10.26</td>
<td>23.22</td>
<td>23.42</td>
<td>35.84</td>
<td>76.72</td>
</tr>
</tbody>
</table>

The main reason for this phenomenon may be as follows. When the temperature is lower than the glass transition temperature of the resin which is usually in the range of 50°C to 120°C (Zhang et al. 2010; Firmo et al. 2015), the resin is in a glassy state and its performance is more or less the same with that at room temperature, so the tensile strength of the GFRP bars change little.

When the temperature is higher than the glass transition temperature but lower than the thermal decomposition temperature of the resin, the resin gradually vitrifies as the temperature increases. However, when the temperature decreases to the room temperature again, the vitrified resin will regain its ability to bond fibers, and the glass fibers can still work together to a certain degree, as such the tensile strength of GFRP bars shows only a slight reduction compared to that of room temperature.

When the temperature is higher than the thermal decomposition temperature of the resin, the resin at the surface of GFRP bars gradually oxidizes and decomposes. Even after the temperature decreases to room temperature again, the properties of the resin cannot be restored, resulting in degradation of the bond between resin and fibres. In addition, the tensile strength of the fibers also gradually decreases as the temperature rises (Bisby and Kodur 2005; Wu 2011), leading to a significant reduction of the tensile strength. When the temperature continues to rise, the resin may ignite and those in the GFRP bars will be in a hypoxic state leading to carbonation and decomposition, resulting in complete softened GFRP bars with sharply reduced tensile strength.

Table 3 shows the change of the elastic modulus of the GFRP bar specimens after exposure to different high temperatures, here the negative numbers indicate an increase of the elastic modulus. It is clear that the change of the elastic modulus is very small after exposure to high temperature within the test temperature range. This is mainly due to the fact that the major factor determining the elastic modulus of FRP bars is the fibers not the resin. Within the test range, the elastic modulus of glass fibers change very little after the high temperature exposure because the test temperatures did not reach the softening temperature of the fibers which is usually within the range of 800°C to 1000°C (Gao et al. 2008b).

Correia et al. thought that if a GFRP composite material is heated up to a temperature above the glass transition temperature (Tg) but below the decomposition temperature (Td) and then cooled down, the decrease in the elastic modulus measured during the heating process can nearly recover to the initial value after cooling. This is because the glass transition is a reversible process and the broken secondary bonds within the polymer molecular structure form again after cooling (Correia, et al. 2015; Bai and Keller 2007)

Table 4 shows the reduction of the ultimate strain after exposure to different high temperatures is show. It can be seen that the ultimate strain of FRP bars was gradually reduced with an increase in the exposure temperature. Note that the ultimate strain is the ratio of the tensile strength to the elastic modulus. As the temperature rises, the tensil
strength decreases but the elastic modulus changes very little, so the ratio of them (being the ultimate strain) decreases with the strength.

<table>
<thead>
<tr>
<th>Exposure temperature</th>
<th>RT</th>
<th>70 °C</th>
<th>120 °C</th>
<th>170 °C</th>
<th>220 °C</th>
<th>270 °C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reduction of ultimate strain, %</td>
<td>0</td>
<td>9.79</td>
<td>14.29</td>
<td>24.67</td>
<td>23.68</td>
<td>35.65</td>
</tr>
</tbody>
</table>

The effect of duration of high temperature exposure

Figures 4-6 shows that at an exposure temperature of 270°C, the tensile strength and the ultimate strain of the GFRP bars show a downward trend with an increase of the duration of exposure. The strength reduction is especially rapid when the exposure time is 0.5 hours (Figure 4). The elastic modulus after high temperature exposure is overall slightly higher than that at room temperature (Figure 5). This suggests that most of vitrification and thermal decomposition of the GFRP bars occur within the first half hour.

CONCLUSIONS

This paper has presented an experimental study on the effects of high temperature exposures on the tensile properties of GFRP bars. The test results have shown that the tensile strength and the ultimate strain of the FRP bars decrease gradually as the temperature increases. When the temperature is lower than 270 °C, the tensile strength reduction is less than 36%. When the temperature reaches to 350°C, the tensile strength of GFRP bars reduced drastically with only 23% of tensile strength at room temperature retained. The effect of high exposure temperature on the elastic modulus of the FRP bars is very small within the test temperature range.

At an exposure temperature of 270°C, most of vitrification and thermal decomposition reaction of GFRP bars is completed within half an hour.
ACKNOWLEDGEMENTS

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TEMPERATURE EFFECT ON THE MECHANICAL PROPERTIES OF FRP-WOOD COMPOSITE

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ABSTRACT

With increasing environmental concern, wood has drawn a global interest as a sustainable structural material. In particular, applying fiber reinforced polymer (FRP) to the tension face of wood beam has been proven as an effective method to achieve strengthening or rehabilitation of wood structures. Although FRP can provide good mechanical enhancement to wood structures, the long-term performance of FRP-wood composite is still a big concern as the deterioration of wood structures is much faster than that of concrete and steel. A comprehensive understanding of the temperature effect on FRP-wood composite is of great importance for a reliable application of FRP strengthening technique. In this paper, the temperature effect on the entire FRP-wood composite has been investigated experimentally. The temperature of environmental chamber for conditioning FRP-wood composites ranges from -5 °C to 50 °C. Three identical FRP-wood specimens have been conditioned and tested for each temperature scenario. It is found that the mechanical properties of FRP-wood composite changes significantly at different temperature levels. The knowledge from this work can act as a foundation for a more reliable design and maintenance strategy for FRP strengthened wood structures.

KEYWORDS

FRP-wood composite, mechanical properties, temperature.

INTRODUCTION

Due to its great environmental benefits, low cost and good mechanical properties, wood is widely used for manufacturing structural component in building industry. However, a relatively low load capacity of wood restricts its application when it is compared to steel and concrete (Bergman et al. 2010). Moreover, wood undergoes biological deterioration with time and it is more vulnerable to detrimental environmental conditions (Calkins 2008). In order to improve the strength and durability of wood structures, strengthening and retrofitting wood structural members should be essentially considered.

Fiber reinforced polymer (FRP), which is made of high strength fibers and a polymer matrix, is widely used in building industry due to its high strength, ease of handling and high resistance to environmental deterioration factors (Lau and Büyükoztürk 2010). Externally bonding FRP to wood is proven as an efficient method to enhance the load capacity of wood structural members (Zhou et al. 2015; Yu et al. 2016). The bonding of FRP to wood is usually achieved by different adhesives, for example: polyurethane, polyester and epoxy (Tam and Lau 2015a). Epoxy has been demonstrated to provide satisfactory bonding performance, and hence it is most common used in FRP externally bonding technique (Tam and Lau 2014).

A number of studies have been conducted to investigated the flexural properties of wood beams strengthened by FRP, including different FRP materials, bond performance between FRP and wood and failure modes, etc (de Jesus et al. 2012; Raftery and Harte 2011; de la Rosa García et al. 2013). Based on these studies, it can be concluded that externally bonding FRP technique is an effective and efficient way to enhance strength and stiffness of flexural wood members. However, the long-term performance of FRP-wood composite is still a big concern as the deterioration of wood structures under environmental conditions are much faster than that of concrete and steel structures (Gunes et al. 2013). In real life applications, wood structures go through four seasons every year and they suffers from significant temperature and humidity changes. Previous study has shown that the mechanical properties and fracture behavior of FRP-wood composite change significantly under different levels of humidity exposure. Water molecules decrease the adhesion energy between epoxy and wood notably (Zhou et al. 2015; Tam and Lau 2015b). Through experimental and molecular dynamic simulation approaches, the interfacial
properties between FRP and substrate material has been demonstrated as the most critical part that governing the mechanical properties of entire composite system and integrity of multi-layer material system under prolonged humid condition (Lau 2012; Tam and Lau 2015c; Lau et al. 2014). However, the temperature effect on FRP-wood composite is still unclear and of great importance for a reliable application of FRP strengthening technique to wood structures.

The objective of this paper is to investigate the effect of temperature on the mechanical behavior of FRP-bonded wood composite experimentally. The temperature conditioning for FRP-wood composites ranges from -5 °C to 50 °C and the duration lasts ten weeks. In each temperature scenario, four identical FRP-wood specimens have been prepared and conditioned. After temperature conditioning, the specimens are tested and the mechanical behavior of FRP-wood composite is obtained.

EXPERIMENTAL PROGRAM

Materials and Samples

The species of wood used in this experiment is the Canadian pine. All wood samples were from one tree to minimize the variability. The wood samples were put into environmental chamber at 20 °C and 10% relative humidity for conditioning (Raftery et al. 2009). After the condition period, an equilibrium moisture content was measured as 13% and the mean density was determined as 510 kg/m³. The tensile and compressive properties of wood along the parallel to grain direction were measured following the Standard ASTM D143. The reinforcement used in this study is unidirectional carbon FRP. The average thickness of FRP is 0.167 mm. The coupon test of FRP sheet was conducted to evaluate the tensile properties in longitudinal direction. The epoxy is used to bond FRP to the tension face of wood beams. The epoxy used here possesses high tensile strength (55 MPa) and high tensile modulus (1.72 GPa) according to the manufacturer, and it can transfer the stress from wood to FRP effectively.

Before bonding FRP to wood, the wood surface should be knife planned to remove impurities to achieve high quality bonding. Meanwhile, the surface of FRP should be cleaned with methylated spirit. After the surface preparation, epoxy is brushed on the wood surface and FRP is placed on the wood beam. The air gaps and excessive epoxy between FRP and wood should be removed though roller. A uniform pressure should be applied on FRP for 24 hours to make sure FRP and wood are well bonded. The FRP bonded wood samples are put in the laboratory condition seven days for curing.

A total of 12 FRP bonded wood beams were prepared. The dimension of wood beam is 420 × 70 × 70 mm. The temperature conditioning in this study contains three levels, i.e. -5 °C, 20 °C and 50 °C. For each temperature scenario, four identical samples were prepared, conditioned and tested. After ten weeks of temperature conditioning, all samples were taken out for four-point bending test. Five strain gauges were bonded to FRP surface of each sample to measure the strain variation during test.

Test Setup

The FRP bonded wood beams were under four-point bending test setup to measure the flexural load carrying capacity. The schematic diagram and photo of four-point bending test setup is shown in Figure 1. The load was applied through a 100 kN load actuator and test was under displacement control at rate of 2.5 mm/min. Linear variable displacement transducers (LVDT) were used to measure the mid-span deflection of FRP bonded wood sample. The strain variation on FRP can be measured by strain gauges bonded to FRP surfaces. During test, the load, mid-span deflection and strain variations were recorded simultaneously by TDS-530 Data Logger.

![Figure 1 (a) Schematic diagram and (b) photo of four-point bending test setup for FRP-bonded wood sample.](image-url)
RESULTS AND DISCUSSIONS

Mechanical Properties of Wood

The tensile and compressive properties of wood are measured as 34.3 MPa (standard deviation: 6.3 MPa) and 38.3 MPa (standard deviation: 4.3 MPa). The tensile elastic modulus and compressive elastic modulus of wood are 9.0 GPa and 7.3 GPa, respectively. The tensile strength of FRP is measured as 3689 MPa with a standard deviation of 330 MPa. The ultimate strain of this FRP is 1.58 % with a deviation of 0.06 %. The elastic modulus of FRP is 234 GPa, which is much higher than that of wood. Based on these data, it can be found that FRP can improve the serviceability limit of wood members significantly. Meanwhile, the tensile strength of wood is smaller than that of epoxy, which implies that epoxy would not fail before wood failure.

Load-deflection Behavior of FRP Bonded Wood Composite

The temperature has a complex effect on FRP bonded wood composite as the thermal expansion coefficient of each component (FRP, epoxy and wood) is quite different. The diversity of thermal expansion coefficient may result in stress concentration along the interface when the entire system is under changing temperature environment, further leading to the failure of interface. The load-deflection relation of FRP bonded wood samples under different temperature conditioning is shown in Figure 2. One representative load-deflection curve is shown for each temperature scenario. The FRP bonded wood sample under 20 °C condition shows the highest load carrying capacity and relatively ductile behavior when it is compared with samples under -5 °C and 50 °C conditioning. When the conditioning temperature increases to 50 °C, a 25.3 % decrease of load carrying capacity can be observed and failure deflection (deflection at ultimate load) also decreases in corresponding load-deflection relation. When the conditioning temperature decreases to -5 °C, the deterioration of load carrying capacity is even severer, up to 38.9 % compared to 20 °C scenario. In order to evaluate the ductility property of FRP bonded wood composite, the equivalent energy elastic-plastic (EEP) estimation of yield point is introduced and ductility ratio is defined in this work (Astm 2005). Based on the typical load-displacement curve, the load of yield point is defined as:

\[ P_y = [\Delta_{\text{failure}} - \sqrt{\Delta_{\text{failure}}^2 - \frac{2w_{\text{failure}}}{K}}] * K \]  

(1)

where \( P_y \) is the yield load; \( \Delta_{\text{failure}} \) is the deflection at failure; \( K \) is the initial stiffness and \( w_{\text{failure}} \) is the energy dissipated until failure. Once the yield load is defined, the corresponding yield deflection \( \Delta_{\text{yield}} \) is determined. Furthermore, in order to determine the ductility level of composite system, the ductility ratio \( \mu \) is defined as:

\[ \mu = \frac{\Delta_{\text{failure}}}{\Delta_{\text{yield}}} \]

(2)

Table 1 summarizes the ultimate load, maximum strain of FRP, energy dissipated until failure, yield deflection and ductility ratio of composite samples under different temperature conditioning. From the ductility ratio in Table 1, we can observe that the ductility ratio decreases slightly when the temperature increases from -5 °C to 20 °C. However, when the temperature keeps increasing to 50 °C, a sharp reduction of ductility ratio, up to 25 % is observed.

From the coupon test of FRP, the ultimate strain for FRP is 1.58 %, while the largest strain of FRP in the four-point bending test is no more than 1.206%, implying that no rupture happens to the FRP during the test. This observation shows that the strength of entire composite system is not limited by tensile strength of FRP. The mechanical properties of wood and the bond performance at the interface are two important parameters governing the mechanical properties of entire system.

<table>
<thead>
<tr>
<th>Temperature conditioning</th>
<th>Ultimate load ( P_u ) (kN)</th>
<th>Max strain of FRP ( \varepsilon_{\text{FRP}} ) (%)</th>
<th>Energy dissipated until failure ( w_{\text{failure}} ) (Nm)</th>
<th>Deflection of yield point ( \Delta_{\text{yield}} ) (mm)</th>
<th>Ductility ratio ( \mu )</th>
</tr>
</thead>
<tbody>
<tr>
<td>-5 °C</td>
<td>30.02</td>
<td>1.206</td>
<td>314.7</td>
<td>2.60</td>
<td>5.34</td>
</tr>
<tr>
<td>20 °C</td>
<td>49.11</td>
<td>1.197</td>
<td>594.9</td>
<td>2.39</td>
<td>6.59</td>
</tr>
<tr>
<td>50 °C</td>
<td>36.70</td>
<td>0.379</td>
<td>146.3</td>
<td>1.95</td>
<td>3.28</td>
</tr>
</tbody>
</table>
Figure 2 The load-deflection curve of FRP-bonded wood sample under -5 °C, 20 °C and 50°C conditioning for 10 weeks.

Strain Variations of FRP

In Figure 3, the ultimate strain variation of FRP at ultimate load level during four-point bending test are shown. From this figure, it can be seen that the stress transfer from wood to FRP is effective in -5 °C and 20 °C conditioning scenarios. However, for the 50 °C conditioning scenario, the strain on FRP layer is very small compared to the other two scenarios, implying that the interfacial bonding performance deteriorated seriously and the stress transfer between wood and FRP is inefficient. Based on this observation, it can be found that 50 °C temperature has a detrimental effect on the interfacial bonding properties.

Figure 3 The ultimate strain variation of FRP bonded wood sample at ultimate load level during four-point bending test.

CONCLUSIONS

In this paper, experimental tests have been performed to investigate the effect of temperature on the mechanical properties of FRP-wood composite. Experimental tests show that the load carrying capacity of FRP-wood composite decreases around 25.3% when the conditioning temperature increases from 20 °C to 50 °C. Furthermore, when the conditioning temperature is lower than freezing point (0 °C), the deterioration of load carrying capacity is much severer, up to 38.9%. The freezing temperature has a negligible effect on the ductility property of composite system, while the high temperature (50 °C) has a detrimental effect on the ductility of entire composite. Furthermore, 50 °C has a negative impact on the interfacial bonding performance between FRP and wood, reducing the strengthening efficiency of FRP material.
As the wood structures suffer from significant temperature variations throughout different seasons, it is necessary to consider the deterioration of FRP-wood composite caused by temperature in design and maintenance of FRP strengthened wood structures. Our current study successfully demonstrates the temperature effect on the mechanical properties of FRP-wood composite system and this knowledge can act as a foundation for a more reliable design and maintenance strategy for FRP strengthened wood structures.

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TEMPERATURE EFFECTS ON CONCRETE SANDWICH PANELS REINFORCED WITH GLASS FIBRE REINFORCED POLYMERS (GFRP)

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ABSTRACT

The thermo-mechanical behaviour of sandwich panels was experimentally investigated. The panels had two external concrete layers reinforced with GFRP rebars and an internal expanded polystyrene insulation layer. These are typical for low bearing panels in façade claddings. The heating condition was adjusted, that the internal GFRP rebars in one concrete layer were exposed, for different times, to increasing temperature higher than the glass transition temperature of the resin. This allowed verifying the modification of the mechanical behaviour, in terms of deformability and loading carrying capacity, of the panels with pre- and post-heating bending tests. As main outcome, the elevated temperature effected significantly the insulation layer and provided considerable reduction of the initial global stiffness and of the load carrying capacity.

KEYWORDS

Glass fibre bars, concrete, sandwich panels, thermo-mechanical loading, mechanical testing.

INTRODUCTION

In the last two decades, several investigations showed the advantages of replacing steel reinforcement with FRP (Fibre Reinforced Polymers) rebars in structural concrete components (Hollaway 2012). Many investigations and applications were focused on the GFRP (Glass Fibre Reinforced Polymers) rebars for their non-corrosive and non-conductive characteristics as well as their high strength, low weight and durability (Micelli et al. 2004). One important field of application of GFRP reinforcement are slender concrete structures, because the concrete cover and, as consequence, the thickness of the concrete members can be reduced. Slender layers of reinforced concrete are frequently adopted as the load carrying members of sandwich panels used as façade panels. External cladding sandwich panels made of concrete usually consist of three layers: thin load carrying concrete layer, thermal insulation and thin facing concrete layer. Sandwich panels are frequently adopted in buildings to exploit their structural and thermal efficiency. Several efforts were dedicated to the understanding and optimization of the mechanical behaviour of sandwich panels with different reinforcements. Some investigations were devoted to the mechanical response of steel free sandwich panels having both reinforcement and connectors of GFRP (see e.g. Schmitt et al. 2014). In spite of the interest on sandwich panels in constructions industry, their durability and in particular, their thermo-mechanical behaviour, is not deeply known and investigated. Some investigations were published for understanding the mechanical behaviour of structural concrete beams (see e.g. Carvelli et al. 2013 and Abbasi et al. 2006) internally reinforced with FRP bars after elevated temperatures exposure. Only few experimental studies were dedicated to sandwich panels subjected to elevated temperatures (see e.g. Hulin et al. 2015). The advantages and the potentialities of concrete elements reinforced with FRP rebars, in particular of sandwich panels, require a safe design (Nigro et al. 2014), when exposed to elevated temperature.

The present experimental investigation aims to give a contribution focused on the examination of the residual mechanical performance of steel free sandwich panels. The present study continues the investigation of the authors detailed in Schmitt et al. (2015), in which the attention was focused on the thermo-mechanical behaviour of thin panels reinforced with GFRP bars, considering the influence of the concrete cover and the external surface of rebars. In the present paper, sandwich panels with two thin GFRP reinforced concrete layers were exposed to increasing temperature and bending loading. The heating condition allowed to reach on the bottom internal GFRP rebars temperatures higher than the glass transition temperature. This does not simulate a real fire exposure, but it is an extreme heating condition for low bearing sandwich panels. The consequences of the thermo-mechanical
loading were assessed measuring the variation of the deformability and of the load carrying capacity of the panels with pre- and post-heating bending tests.

MATERIALS AND SAMPLES

The geometry of the sandwich panels is detailed in Figure 1. The length and the width are the maximum for the available heating device. The two concrete layers of thickness 4 cm were reinforced with E-glass fibres reinforced polymer (GFRP) bars. Concrete cover of 10 mm was adopted for the GFRP rebars, named commercially Schöck ComBAR®. The configuration of the reinforcement consists of longitudinal and transverse rebars of nominal diameter 8 mm. In between the two concrete layers, a 6 cm expanded polystyrene insulation layer (typical for façades, named commercially Joma EPS 040 WDV) was placed to reproduce a sandwich panel adopted as external cladding. Point-shaped GFRP connectors of diameter 12 mm were used to link the three layers. They had the same shape of the external surface of the reinforcement, as in Figure 1.

![Figure 1 Specimen geometry and reinforcement]

The number of tests is summarized in Table 1. The GFRP rebars were produced by pultrusion technique using vinyl ester resin with glass transition temperature (T_g) of about 180 °C, according to the producer. They have external ribbed surface, cut into the bar after curing (Figure 1). The main mechanical properties in the longitudinal direction of the rebars are the tensile strength of ≈ 1000 MPa and the elastic modulus of ≈ 60 GPa. The mix for concrete contains for m³: 205 kg of water; 485 kg of cement CEM 42.5 N; 729.6 kg of aggregates 0/2 mm; 891.7 kg of aggregates 2/8 mm; 2 kg of plasticizer (ACE 30). During casting, some specimens were prepared to measure the average mechanical properties of concrete: compressive cubic strength 49.9 MPa, compressive elastic modulus 23.4 GPa and tensile strength 3.1 MPa.

EXPERIMENTAL SET-UP

First phase: pre-heating mechanical response

![Figure 2 Test set-up and external instruments position of first and third phase]

Table 1 Numbering of samples and some heating features

<table>
<thead>
<tr>
<th>Panel ID</th>
<th>Heating rate [°C/min]</th>
<th>Heating time [min]</th>
<th>Exposition time [min]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-heating loading</td>
<td>3 and 2</td>
<td>20</td>
<td>16</td>
</tr>
<tr>
<td>Heating and Post-heating loading</td>
<td>4</td>
<td>50</td>
<td>6.4</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>20</td>
<td>20</td>
</tr>
</tbody>
</table>
Two panels (named 1 and 2) were loaded with supports span of 140 cm and loads span of 50 cm (see Figure 2) to have the reference mechanical behaviour before heating. The bending loading was applied quasi-statically with an initial speed of 0.5 mm/min for initial cracks monitoring and then of 2 mm/min up to failure, using a jack with a load cell of 50 kN. The width of cracks was measured, maintaining constant the displacement of the jack for less than one minute, at each load step of 2.5 kN. During loading, the deflection of the specimen was measured by five transducers (LVDT), one in the mid span, two 35 cm beside both supports and two at the supports position (see Figure 2).

Second phase: thermo-mechanical loading

In the second phase, quasi-static four-points bending loading was first imposed with same scheme as in the first phase. At the maximum imposed load, the heating device started increasing the temperature on the full bottom surface of the specimen, which was exposed for the time listed in Table 1. The heating on the bottom surface of the samples was applied by the device in Figure 3a. This is an electronically controlled oven with maximum temperature of more than 1000 °C. The temperature was controlled by a thermocouple in the centre of the device.

Figure 3 Heating device (a); Test set-up and external instruments position of second phase (b)

The temperature on the bottom and top surface of the specimen was measured by two thermocouples TC-B and TC-T. Moreover, in two panels (named 3 and 4) one thermocouple (TC-1) was placed mid-span on the bottom of the central rebar to monitor the internal temperature at the bar level. In the panel 5, four more thermocouples (TC-5 to TC-8) were placed at the level of both interfaces between the concrete and the insulation layer. Those allowed the monitoring of the temperature in the thickness of the panel during heating (see Figure 3b).

After the application of the loading, the temperature inside the device was increased from room temperature to the maximum of almost 320 °C for panel 3 and 4 and almost 400 °C for panel 5, with different heating rates. When the temperature in the centre of the device reached the desired maximum, it was maintained constant until the end of the heating phase. The heating phase was about 60 min for panels 3 and 4 and 80 min for panel 5.

The maximum imposed temperature is not as in some real applications of external cladding (e.g. on façade panels the temperature could be below 100 °C in very warm latitudes). It was considered to investigate the thermo-mechanical behaviour under extreme conditions. The maximum temperature in the device was imposed to reach at the rebars level a temperature higher than the glass transition temperature. During loading and heating the deflection of the specimen was measured by five transducers (LVDT) as shown in Figure 2.

Third phase: post-heating mechanical response

In the third experimental phase, after complete cooling, the specimens were quasi-statically loaded using the same four-points bending setup and procedure of the pre-heating tests (see Figure 2) at room temperature. This allowed comparing the mechanical behaviour of the panels, pre and post the thermo-mechanical conditioning.
RESULTS AND DISCUSSIONS

First phase: pre-heating mechanical response

Four points bending tests at room temperature of two sandwich panels (named 1 and 2) provided the reference mechanical behaviour up to failure. The load vs. mid span deflection curves (Figure 4a) show a very similar behaviour of both panels. The two branches shape is quite similar to those recorded for the same load condition of sandwich panels with different reinforcements (Schmitt et al. 2014). An initial branch with the maximum stiffness of the panel up to a load level (= 19 kN), for which the propagation and diffusion of cracking generate a high decrease of stiffness, in the second branch, leading to failure. Looking more in detail the first branch of the two curves in the low range of load, they can be considered linear up to a load level of almost 4 kN (Figure 4b), then the curves had a variation of slope meaning initiation of cracks. This was considered as reference level for loading in the second phase. Both panels failed under compression of the concrete (see Figure 4c).

![Figure 4 Load vs. mid span displacement: complete curves (a); curves in low range load (b); Failure mode (c)](image)

Second phase: thermo-mechanical loading

In the second experimental phase, the thermo-mechanical loading was subdivided in three consecutive steps: mechanical loading; heating with different rates; maintaining the temperature constant in the centre of the device for different exposition times. The bending loading was applied quasi-statically increasing the load in almost 2 minutes up to the maximum resultant of 5 kN (specimen own weight is not included) to impart an initial cracking before heating. The maximum load was estimated assuming the results of the room temperature bending tests in the first phase (see Figure 4b). The heating rate and the exposition time for the three panels (named 3, 4 and 5) adopted in the second phase are listed in Table 1. The effect of two heating rates (20 and 50 °C/min) and two exposition times (60 and 80 min) were investigated.

![Figure 5 Temperature vs. time of panels 3, 4 and 5 on the centre of top (TC-T) and bottom (TC-B) (a) and at mid span on central bar (TC-1) (b); LVDTs displacement vs. time during heating of panel 5 (c)](image)

The temperature on the bottom and top surface of the specimen was continuously measured in the mid span (see Figure 5a). During the exposition time, the temperature on the top (unheated side) was below 30°C, while on the bottom (heated side) there were differences; panel 3 with the lowest heating rate and heating time had a maximum temperature of 225 °C, panel 4 with the highest heating rate and the lowest heating time had a maximum temperature of 260 °C and for panel 5 with the lowest heating rate and the highest heating time a maximum...
temperature of 292 °C was recorded. The diagram in Figure 5b shows the evolution in time of the temperature recorded on the bottom rebar (TC-1). The protection of 1 cm concrete cover allowed to enable maximum temperatures of the rebars in the bottom layer in the range of 195±260 °C. The temperature of the rebars after 40 to 50 min was higher than the resin glass transition temperature, according to the heating rate, and then they were exposed to increasing temperature, higher than T_g, for almost 10, 20 and 30 min in panel 3, 4 and 5, respectively. The measurements of the temperature at the interfaces of the insulation and the two concrete layers were recorded for panel 5 by four thermocouples (TC-5 to TC-8), fixed on two connectors (Figure 3b). The maximum temperature after the heating exposition at the interface of the insulation and bottom concrete layer was almost 181 °C, while at the top interface it was 41 °C. When the temperature at the bottom side of the insulation reached 90 °C, the material underwent a continuous degradation until the evaporation of a large portion of the insulation layer (see Figure 6 after the heating phase). The continuous recording of the LVDTs provided the evolution of the displacement as illustrated in Figure 5c for panel 5. In particular, the initial loading lead to a mid-span displacement of all panels of almost 1.5 mm, while the considered heating rates and exposition times imparted different deformation of the panels. The global deformation of the panels measured after the exposition phase does not seem dependent on the considered heating rates and times, but is proportional to the maximum temperature imposed on the bottom surface. The mid-span displacement after heating of panel 5 is 30 % higher than that of panel 3. After heating and unloading, the maximum residual deflection was 8.5 mm for panel 5. The thermo-mechanical loading of the second experimental phase imparted a cracks pattern on the bottom surface of the panels. The main distribution of the cracks is located, as expected, in the central part of the panels with the constant maximum bending moment. Few longitudinal cracks were observed, mainly in the two external portions close to the supports.

![Figure 6](image_url) Expanded polystyrene layer of panel 4 after heating phase

### Third phase: post-heating mechanical response

The influence of the thermo-mechanical exposition on the mechanical response of the panels was measured in the third experimental phase with four points bending loading up to failure at room temperature. The global response of the panels is detailed in Figure 7a comparing the pre- and post-heating load vs. mid span deflection curves. The different behaviour of the panels is evident, observing the failure loads. Panel 3, subjected to the lowest temperature, had a reduction of the maximum load of about 8 %, while panel 4 and 5, which were heated with higher temperatures, had a failure load 27 % and 30 % lower than the reference panels. This highlights that the reduction of the failure load is not dependent on the considered heating rates and times, but mainly on the maximum temperature recorded on the bottom surface. The influence of the thermo-mechanical exposition is also measurable comparing the initial stiffness of the panels (see Figure 7b), which is defined as the slope of the segment passing through the two points of a curve at load 1 and 4 kN. The stiffness of the panel had a reduction of 22 %, 52 % and 59 % for panel 3, 4 and 5, respectively. The comparison shows, that the higher the applied maximum temperature, the higher is the reduction of the stiffness, regardless of the heating rate and time.

![Figure 7](image_url) Comparison of the pre- and post-heating bending load vs. mid span displacement (a); initial stiffness of the panels (b); rebars in the mid span of the bottom concrete layer after failure in third phase (c)
Two reasons can be attributed to the discussed reduction of the mechanical properties. The first is the damage imparted in the concrete layers (mainly transversal cracks) during the thermo-mechanical loading. In fact, the rebars had, after removing of concrete, the shape in Figure 7c. The bars were not damaged and not broken. They had still a good adhesion with concrete and their external surface was not apparently modified by the elevated temperature. Therefore, the observed level of the damage imparted could have only a marginal influence on the reduction of the mechanical features of the panels. The second reason is the degradation and lack of large portion of the insulation after heating. This is the main responsible of the significant reduction of the mechanical response of the panels. The absence of insulation layer extremely reduced the bond between the top and the bottom layer. Therefore, the structural role of the insulation (Lameiras et al. 2013; Schmitt et al. 2014; Müller et al. 2012) in transferring the shear forces between the concrete layers (sandwich load carrying action) and in giving a contribution to the bending stiffness and load carrying capacity of the panels was completely lost.

CONCLUSIONS

The experimental research was focused on understanding the thermo-mechanical response of steel free sandwich panels reinforced with GFRP rebars. The influence of two parameters was investigated: the heating rate and the exposition time. The limited number of tests does not provide statistically significant results, but they give a clear trend of the mechanical behaviour under the considered conditions. The main outcomes of the research are:

- The imposed thermo-mechanical exposition, up to a maximum in the concrete of 292 °C, created some transversal and longitudinal cracks on the bottom surface of the panels. The GFRP bars were not damaged, had still a good adhesion with concrete and their external surface was not apparently modified.
- When the temperature at the bottom of the expanded polystyrene was about 90 °C, the insulation started melting and the material had a continuous degradation until the evaporation of a large portion of the layer.
- The significant reduction of the mechanical properties (in terms of failure load and stiffness) was mainly related to the max temperature on the bottom surface, regardless of the heating rate and exposition time.
- The main responsible of the considerable reduction of the mechanical response was supposed to be the degradation and lack of large portion of the insulation after heating. The absence of insulation layer reduces the load transfer capacity from the top to the bottom concrete layer (sandwich load carrying action).

The obtained results demonstrate the excellent mechanical behaviour of the thin reinforced concrete layers and in particular of the GFRP rebars exposed to a range of temperature higher than the glass transition temperature of the resin. Moreover, the importance on the global mechanical behaviour of the insulation layers was highlighted to avoid the degradation of the mechanical features after exposition to elevated temperature.

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REFERENCES


LONG TERM PERFORMANCE OF GFRP BARS UNDER THE COMBINED EFFECTS OF SUSTAINED LOAD AND SEVERE ENVIRONMENTS

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ABSTRACT
This paper investigates the durability of Glass FRP (GFRP) reinforcement exposed to severe environments and subjected to different levels of sustained load. The test matrix was designed to include 92 GFRP specimens exposed to temperatures varying from 20°C to 60°C for 0, 42, 90 and 270 days. A total of 69 specimens were embedded in concrete and submerged in water, whilst the remainders were submerged in alkaline solution with a pH of 12.7. Tensile tests were performed on reference samples and on all conditioned specimens to determine their initial and residual strength and stiffness.

The effect of different exposure conditions and sustained stress on the long term properties of the tested bars is examined and discussed in detail. Exposure to elevated temperatures seems to play a key role in triggering and accelerating the development of critical degradation mechanisms. Moreover, the higher moisture absorption rate corresponds to high reduction on tensile strength and transverse mechanical properties. The effects of both levels of sustained stress (equivalent to 3000 με and 5000 με) seem to be negligible.

KEYWORDS
Composites, durability, GFRP, severe environment, sustained loading.

INTRODUCTION
Over the past three decades, the search for valid alternative to steel reinforcements in concrete structures to prevent corrosion issues has received a lot of attention mainly due to the high cost of maintenance generally associated with critical infrastructure (e.g., bridge decks) (Nkurunziza et al., 2005, Benmokrane and Mohamed, 2013, Almusallam et al., 2013, Demis et al., 2007, Micelli and Nanni, 2004, Dejke, 2001, Robert et al., 2010, Mufti, 2007). Among all different types of Fibre Reinforced Polymer (FRP), Glass FRP (GFRP) bars became the most appealing to civil engineers due to their good mechanical performance and low production costs in comparison to the other FRP bars. However, while the expected service life of an RC structures spans from 50 to 100 years, no reliable information on the long-term performance of in-service GFRP RC structures is yet available.

The physical and mechanical properties of FRP bars can degrade over time due to fatigue, creep, exposure to moisture, high temperature, alkalinity or ultraviolet rays (fib TG 9.3, 2007). The durability of FRP and their long term properties are generally examined through the implementation of accelerated ageing techniques consisting of laboratory tests during which specimens are subjected to mechanical or chemical conditioning along with elevated temperatures to accelerate the onset and development of the degradation processes (Bakis et al., 2002). The exposure conditions are usually selected so as to reproduce the in-service conditions of GFRP bars (i.e., embedded in concrete (moist and alkali environment) under sustained stress).

While previous research mainly focused on studying the effect of individual degradation parameters, this research aims to investigate the long-term behaviour of GFRP bars subjected to couple mechanical and environmental effects, thus representing more closely the in-service conditions of FRP RC.
EXPERIMENTAL PROGRAMME

The experimental programme was designed to study the development of mechanical and physical properties of GFRP bars exposed to alkali (concrete and alkali solution) and moist (tap water) environments as well as sustained stress. The test matrix included 92 GFRP bars with a nominal diameter of 8 mm, made of continuous E-Glass fibres impregnated in a vinyl ester resin and manufactured using the pultrusion process.

The specimens were categorised into four Groups. The unconditioned reference specimens were in Group 1, while Group 2 consisted of stressed and unstressed bare bars conditioned in alkaline solution or tap water at 20, 40 and 60°C. Stressed and unstressed bars embedded in concrete were kept in the laboratory at ambient temperature (Group 3) and immersed in tap water at 20, 40 and 60°C (Group 4). Specimens in Groups 2 and 4 were conditioned for 1000, 2000 and 6480 hours.

The alkaline solution was made of 118.5 g of Ca(OH)$_2$, 0.9 g of NaOH and 4.2 g of KOH in 1 litre of deionised water as recommended by (ACI 440.3R, 2004). The pH level of this solution was periodically controlled and kept constant at about 12.7, which is a representative value for a mature concrete pore solution.

In real applications, structural elements are usually subjected to a service load, which in turn results in a given value of stress being applied to the FRP reinforcement. The stress sustained by the FRP reinforcement can induce micro cracks in the matrix and, in the presence of moisture, promote absorption of the surrounding chemical materials. The chemical interaction between hydroxyl ions and the FRP constituents can consequently increase the rate of the degradation processes in both polymer matrix and fibres. So as to examine this phenomenon, a sustained stress inducing a tensile strain of 3000 με was applied to the FRP bars as recommended by design standards of practice (ACI 440.1R, 2006, CAN/CSAS806-2012). A higher stress inducing a strain of 5000 με was also examined in this investigation to assess the effect of less stringent serviceability limits (e.g. larger crack widths and deflections) on the long term properties of the GFRP specimens.

Two different methodologies were employed to apply a sustained stress equivalent to 3000 με and 5000 με during the whole aging process. For moisture absorption and inter-laminar shear (ILSS) tests, pairs of specimens of Group 2 were tied together at the two ends while a wedge was interposed at mid-length to impose the required curvature and apply the desired stress (Figure 1a). The length of the bars and the size of the wedge were determined according to elastic theory calculations. Conversely, for tensile tests, specimens embedded in concrete (Groups 3 and 4) were placed in a custom made, post-tensioning steel frame equipped with a calibrated spring (Figure 1b).

In this configuration, the aforementioned levels of stress and strains were ensured by simply compressing the spring to the calculated length. Demec points were also mounted on the specimens and used to verify the induced, initial level of strain and monitor strain variation during the whole duration of the conditioning phase.

![Figure 1 Application of sustained stress for specimen of Group 2 (a) and Group 3 and 4 (b). Both pictures were taken after conditioning.](image)

DURABILITY EVALUATION OF GFRP BARS UNDER SUSTAINED STRESS

Characterization of mechanical properties

The mechanical properties of the GFRP bars have been investigated after different durations of exposure, ranging from 1000 to 6480 hrs. The measured mechanical properties were tensile strength, inter-laminar shear strength and modulus of elasticity. The direct tensile tests were performed according to ACI 440.3R-04. Prior to any conditioning, all specimens were fitted with appropriate end anchors filled with epoxy resin that could be mounted directly in the tensile testing machine or in the bespoke post-tensioning rigs (figure 2). Before testing, the bars embedded in concrete were carefully extracted using a hammer and chisel to avoid any damage to the bar. The bars were instrumented with an electronic extensometer placed along their central portion. The extensometer had gage length of 51 mm and an accuracy of 0.025 mm (figure 3). The tests were carried out in displacement control using a universal testing machine. The displacement rate was 1.5 mm/min. The applied load and bar elongation were recorded during the test using a control data acquisition system.
The inter-laminar shear tests were performed according to ASTM (D7028, 2016). The specimens were obtained from FRP bars exposed to a combination of temperature, environments and sustained load, and had a length sufficient to guarantee a clear span of 30 mm and adequate overhanging portion. The span was selected in order to obtain the desired inter-laminar shear failure. The tests were performed in displacement control using a Shimadzu testing machine with a total load capacity of 10 kN. The displacement rate of the crosshead was set to 1.0 mm/min. The ILSS test apparatus is shown in figure 3.

![Figure 2 Sketch illustrating the geometry of a typical tensile test specimen](image)

![Figure 3 Tests set up for tensile strength (left) and ILSS (right)](image)

**Characterization of physical properties**

Physical and microstructural analyses were performed on representative samples, including water absorption, glass transition temperature and high resolution imaging. Moisture absorption measurements were carried out periodically according to ASTM (D7028-07, 2015) and the weights of the GFRP samples were measured monthly to examine the increase due to fluid uptake. The glass transition temperature was measured performing a set of Dynamic Mechanical Analysis (DMA) according to ASTM (D570-98, 2010) using a Mettler Toledo DMA/SDTA861e machine. Moreover, the microstructural analysis was conducted using Scanning Electron Microscopy (SEM) with a high magnification microscope (Phillips XL30 SEM at 20 kV).

**TESTS RESULT AND DISCUSSION**

Table 1 summarizes the results of the mechanical tests along with the measured levels of water absorption after a given duration of exposure to the predetermined environments. The average tensile and ILSS strengths obtained from testing three samples (5 for the control specimens) are reported along with the coefficient of variation and percent rate of retention.

The variation in tensile strength and ILSS for the tested specimens is reported in the following and discussed along with some preliminary observations drawn from the tests used to characterize the physical properties.
Table 1 Tensile strength, ILSS and water absorption tests results

<table>
<thead>
<tr>
<th>Time (hrs)</th>
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<th>Sustained Strain (με)</th>
<th>Tensile Strength</th>
<th>ILSS</th>
<th>Water absorption</th>
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<td></td>
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<td>Ret (%)</td>
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<tr>
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<td></td>
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Figure 4 Tensile Strength Retention and moisture absorption of unstressed GFRP bars vs duration of exposure

*Con concrete, W water, TEN tensile strength, Abs Absorption rate
Figure 5 Tensile Strength Retention and moisture absorption of stressed GFRP bars vs duration of exposure
*Con concrete, W water, TEN tensile strength tests, Abs Absorption rate

Tensile strength retention

All specimens failed by rupture of the bar and failure initiated within the free length of the specimens without any noticeable slip taking place within the anchorage length. The average tensile strength of the control samples was 1542±28 MPa. It is clear that the tensile strength of the conditioned specimens reduced as the temperature and conditioning time increased. As expected, the highest degradation rate in tensile strength was found for the specimens exposed to a temperature of 60°C. However, the two imposed levels of sustained stress (corresponding to an induced strain level of 3000 με or 5000 με) did not seem to affect significantly overall strength retention.

Inter-laminar shear strength (ILSS)

ILSS is strongly related to the properties of the resin matrix. A summary of the tests results on the ILSS of the tested specimens is reported in table 1 and shown in figure 6, along with moisture absorption to illustrate the effect of moisture uptake.

Figure 6 Inter Laminar Shear Strength retention and Moisture absorption rate for the tested environments
*W in water, WS in Water stressed specimens.
All specimens showed the desired failure mode, which was characterized by the development of a crack perpendicular to the direction of application of the load. However, a combination of horizontal and vertical cracks developed in some of the conditioned specimens, possibly as a result of a relatively high water absorption. In summary, figure 6 shows that higher temperatures of exposure result in a higher reduction in ILSS. Moreover, higher moisture absorption rates correspond to lower ILSS.

**Tests result of physical properties**

The tests results showed that higher temperatures of exposure correspond to an increased weight gain. Moreover, higher moisture absorption rates corresponded to higher reduction in tensile strength and transverse mechanical properties (figures 4, 5 and 6). Unexpectedly, the stressed specimens experienced lower absorption rate than the unstressed counterparts. This may be attributed to the method that was used to induce the desired induced strain (i.e. imposing the required curvature via elastic bending) and the closing of micro cracks within the compression zone of the specimens. No significant microstructural changes were observed from the SEM images. On the other hand, DMA showed a slight reduction in Tg for the samples conditioned in alkaline solution for 2000 hrs.

**CONCLUSIONS**

In this paper an experimental program aiming to quantify the deterioration of GFRP bars in the concrete environment has been reported. Based on the test results presented here the following conclusions can be drawn:

- Among all parameters examined, temperature and moisture were found to be the most important factors affecting the degradation process.
- No significant change in the composite microstructure was observed after 90 days of exposure to the harshest environment (alkaline solution at 60°C).
- The application of sustained stress did not seem to affect significantly the residual mechanical properties.

**REFERENCES**


TENSILE PROPERTIES OF CFRP PLATES UNDER HIGH STRAIN RATES

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ABSTRACT

This paper presents the results of experimental investigations of the tensile properties of different types of CFRP plates. Coupon specimens of low, normal and ultra-high modulus CFRP plates were prepared and tested under tensile loading. Four different strain rates were used, including one low strain rate and three high strain rates. Two methods of capturing strain were used: image correlation photogrammetry and foil strain gauges. The results show a significant increase in the tensile strength and elastic modulus. The CFRP coupons were prepared and tested in accordance with the ASTM3039-08 standards.

KEYWORDS

CFRP plates, high strain rate, mechanical properties, CFRP tensile strength.

INTRODUCTION

Carbon fibre reinforced polymers (CFRPs) are commonly used in structural rehabilitation. CFRP is used to strengthen steel structures such as bridges, buildings, trusses, harbour structures, which are usually subjected to different types of loadings with different velocities. A large number of studies have focussed on the strengthening of steel members with CFRP composites, including both experimental and analytical investigations, in order to better understand the behaviour of such applications. The properties of CFRP composites are usually provided by the manufacturer, and these material specifications are measured under static tension or compression loads according to the standards used. These properties can be considered in strengthening applications that are statically loaded. When it comes to high load rates applications, the material properties might differ from the quasi-static loadings. Therefore, there is a problem with studying the bond strength and design calculations without investigating the composites material properties under the desired velocities. A number of experimental investigations have been conducted to study the tensile mechanical properties of various types of FRP under different variables. However, static and dynamic tensile loading are the main loadings used in the research literature. Jacob et al. (2004) presented a review of the strain rate effect on the material properties of FRP composites. The main focus of Jacob et al.’s paper is to review research on the effect of velocity to gauge length ratios, and the effect of these ratios on different mechanical properties of FRP under different applied loads.

There are different methods of performing impact testing on FRP composite materials. The Charpy pendulum, drop mass, hydraulic instruments and split Hopkinson pressure bar are techniques available to obtain the desired strain rate, and each of these techniques can achieve a range of strain rates. These methods are summarised by Deshpande (2006). Harding and Welsh (1983) modified the split Hopkinson pressure bar technique for testing materials with unidirectional fibres such as FRPs under high tensile loading rates. Their research showed that the mechanical properties of glass fibre reinforced polymers (GFRPs) is significantly affected by strain rate for both the 0° and 45° fibre directions. Other studies have focussed on this type of testing using different types of FRP and different strain rates (Adams and Adams, 1989; Adams and Adams, 1990; Lifshitz and Leber, 1998; Barré et al., 1996; Hou and Ruiz, 2000; Majzoobi et al., 2005; Fernie and Warrior, 2002).

This paper presents the mechanical properties of CFRP laminate under high strain rates. Previous studies have been conducted by the authors on the bond properties between CFRP laminate and steel members under high strain rates (Al-Mosawe et al., 2016a; Al-Mosawe et al., 2015; Al-Mosawe et al., 2016b). The properties of three types of CFRP laminates are investigated in this paper: tensile strength, ultimate strain and elastic modulus. Four velocities were used starting from quasi-static up to 5 m/s.
MATERIALS

Three types of CFRP laminates were utilised in this project: low, normal and ultra-high modulus. The tensile strength (MPa) and elastic modulus (GPa) for each type of CFRP are 2800, 165; 2800, 205; and 1500, 460 respectively, as provided by the manufacturer’s specifications. A list of the other properties is shown in Table 1.

<table>
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<tbody>
<tr>
<td>Fibre modulus (GPa)</td>
<td>165</td>
<td>205</td>
<td>460</td>
</tr>
<tr>
<td>Thickness (mm)</td>
<td>1.4</td>
<td>1.4</td>
<td>1.2</td>
</tr>
<tr>
<td>Tensile stress (MPa)</td>
<td>&gt;2800</td>
<td>&gt;2800</td>
<td>1500</td>
</tr>
<tr>
<td>Density (g/cm³)</td>
<td>1.6</td>
<td>1.6</td>
<td>1.82</td>
</tr>
<tr>
<td>Ultimate elongation %</td>
<td>&gt;1.6</td>
<td>&gt;1.35</td>
<td>0.3%</td>
</tr>
</tbody>
</table>

SPECIMEN PREPARATION AND TEST SET-UP

Unidirectional carbon fibre reinforced polymer laminate CFK 150/2000, CFK 200/2000 and MBrace 460/1500 were used in this study. The three types were tested under static and dynamic load to investigate the effect of impact load on their mechanical properties. Five samples for each CFRP type were prepared for each test (one static and three dynamic tests), and the samples were prepared in accordance with ASTM: D 3039/D 3039M-08. Steel tabs were glued at both ends of the specimen as shown in Figure 1, to reduce the stress concentration and distribute the force uniformly to the CFRP specimens. These tabs were sand-blasted and cleaned with acetone, a thin layer of adhesive was applied on their surface, and then they were attached at both ends of the CFRP laminate on both sides. The far end surfaces of the CFRP were also cleaned with acetone, to ensure they were free of any dust or contaminants.

![Figure 1 (a): specimen for dynamic test, (b) specimen under quasi static load; (c) the image correlation technique system](image)

The procedure for specimen preparation was similar for static and dynamic testing, apart from the specimens’ dimensions; the gauge length was 150mm for the static and 50mm for the dynamic testing. This difference in gauge length was used to achieve the desired strain rate values.

Four strain rate values were used, starting from quasi-static tests at 2.22×10⁻⁴ s⁻¹, to impact tests at 67, 86 and 100 s⁻¹. MTS 250kN machine was used for the quasi-static testing, while a drop-mass rig was utilised to test the coupons under impact load. The desired strain rates for impact testing were achieved by dropping a mass of 156kg from different heights (0.575, 0.975 and 1.3m). For the impact tests, a strain gauge was mounted at the centre of the coupon to measure the strain development while testing (see Figure 1a). However, for the quasi-static testing, two methods were used to measure the strain: a conventional strain gauge and photogrammetry image correlation. The photogrammetry image correlation technique (see Figure 1c) has been shown to be efficient in previous research by comparing its strain readings with the strain gauge reading at the same location in CFRP-steel double strap joints (Al-Mosawe et al., 2015). Following the manufacturer’s instruction for using this technique, all coupons were painted white along the gauge length, and then each coupon was painted with black dots using a fine marker, as recommended in the manual of the VIC-3D correlated solution camera. The camera captured the strain that developed on the coupons while loading, Figure 1b shows painted specimen under loading.
RESULTS AND DISCUSSION

It is common for the actual properties of FRP to differ slightly from the manufacturers’ properties, due to the fact that the manufacturers provide the design properties or the minimum mechanical properties of the composite materials. Below are the details of the experimental results for all types of CFRP under the different loading rates.

Effect of high strain rate on stress-strain relation

The ultimate stress varies among the different strain rates, and the effect of high strain rate on the stress-strain relationship is shown in Figure 2 below.

![Figure 2 stress-strain relationship for CFRP laminates](a) 2.22×10⁻⁴ s⁻¹, (b) 67 s⁻¹ (c) 86 s⁻¹, (d) 100 s⁻¹

As shown in the figures above, a sudden failure occurred (at the point where the plots end), indicating that CFRP laminate is an elastic brittle material. The stress-strain behaviour is similar for all strain rates with slightly different values.

Effect of high strain rate on tensile elastic modulus

The results show that there is a significant impact on the tensile modulus of all CFRP types under high strain rates, as shown in Figure 3. The elastic modulus increased by up to 20% under high strain rates. The influence of high strain rates on the elastic modulus of CFRP was varied among the different research (Lifshitz and Leber, 1998; Harding and Welsh, 1983; Adams and Adams, 1990; Hou and Ruiz, 2000; Al-Zubaidy et al., 2013). The variations in the influence rate among all studies can be attributed to the type of FRP used and the strain rate values.

Effect of high strain rate on ultimate tensile stress

Figure 2 shows the linearity behaviour of CFRP composite materials, while the ultimate strength of all CFRP types significantly increased under the high strain rates. Figure 4 shows that the tensile strength increased by up to 55% under high strain rates. This percentage is higher than that found by the other researchers mentioned above, and
this variation may be attributed to the type of CFRP used. In addition, the type of matrix used to laminate fibres may also have some influence on these rates.

**Effect of high strain rate on ultimate longitudinal strain**

The effect of high strain rates on the ultimate longitudinal strain was significant for low and normal modulus CFRP. However, there was an insignificant increase in the ultimate strain for ultra-high modulus CFRP, as shown in Figure 5. The lack of understanding of the effect of high strain rates on longitudinal strain is due to the limited number of studies focusing on this. In the present research, the strain rate sensitivity to fibres is the reason for this behaviour.

![Figure 3: Effect of strain rate on the Young’s modulus of CFRP](image)

![Figure 4: Effect of strain rate on the ultimate tensile strength of CFRP](image)

A comparison of the current test results of CFRP laminates and previous research with CFRP sheet are carried out in this research. The comparison is made with the study held by Al-Zubaidy et al. (2013), normal CFRP sheet was used in this research to investigate its engineering properties under a range of strain rates. Three of the strain rate values are matching those used in this research, so it is worth to plot the results in figures to have better understanding. Figure 6a, 6b and 6c are showing the comparison of the strain rate vs the elastic modulus (GPa), Tensile strength (MPa) and ultimate strain respectively. The results of this research agreed very well with those found in CFRP sheet. The behaviour of the increase in material properties with the increase of strain rate can be attributed to the strain rate dependency of CFRP fibres, where FRP laminate has about 66% fibres and the rest is matrix as mentioned in the manufacturer properties.
Figure 5 Effect of strain rate on the ultimate tensile strain of CFRP

Figure 6 Comparison of the material properties between CFRP laminate and CFRP sheet under high strain rates.

CONCLUSION

The mechanical properties of three types of CFRP: low, normal and ultra-high modulus, were investigated. The properties were measured under high strain rates to study their influence on tensile strength, ultimate strain and elastic modulus. Four strain rates were used in this study: $2.22 \times 10^{-4}$, 67, 86 and 100 s$^{-1}$. The results show a significant increase in the engineering properties under high strain rates, the tensile stress being increased by up to 55%, while the elastic modulus and the ultimate strain increased by up to 20% and 35% respectively under high strain rates. Similar behaviour of material properties was observed for CFRP laminates and CFRP sheets.

ACKNOWLEDGMENTS

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the Smart Structures Laboratory at Swinburne University of Technology and the structural laboratory at Monash University for their assistance.

REFERENCES


EFFECT OF SHEET VOLUME ON IMPACT RESISTANT CAPACITY OF RC BEAM STRENGTHENED WITH AFRP SHEET

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ABSTRACT
In this study, in order to investigate effects of the sheet volume on impact resistant capacity of the RC beams strengthened with Aramid Fiber Reinforced Polymer sheet (AFRPs), weight-falling impact tests were conducted taking sheet volume and falling height of the weight as variables. In addition, Poly-Ethylene Telephthalate FRP sheet (PFRPs) was also used to absorb the impact force and/or restrain rupturing of the AFRPs combining with AFRPs. An elastic modulus of PFRPs is less than one tenth that of AFRPs, but the ultimate strain is larger than four times that of AFRPs. The results obtained from this experiment are as follows: 1) the mid-span deflection of the RC beams can be reduced due to bonding AFRPs; 2) the impact resistant capacity of the RC beams can be increased corresponding to an increase of sheet volume; and 3) the axial strain concentration and rupturing of AFRP sheet can be restrained due to increasing sheet volume and/or using PFRPs combining with AFRPs.

KEYWORDS
RC beams, AFRP sheet, PFRP sheet, sheet volume, impact resistant capacity.

INTRODUCTION
In recent years, various Fiber Reinforced Polymer sheet (FRPs) bonding methods have been applied for retrofitting and/or strengthening existing Reinforced Concrete (RC) structures. On the other hand, impact resistant structures also need to be strengthened, because some structures have been deteriorated under severe environmental conditions. However, an effective strengthening procedure for this kind of structures has never been established yet. In our previous works, Aramid FRP sheet (AFRPs) bonding method has been proposed as one of strengthening method for impact resistant structures, because AFRPs has many advantages such as light weight, high tensile strength, flexible, and ductile properties.

Until now, our research group conducted falling-weight impact test of the RC beams strengthened by means of AFRP sheet bonding method (Kurihashi et al. 2015). In consequence, it is confirmed that the deflection of the beams and the crack width around the loading point can be reduced by applying the proposed strengthening method. However, when the input impact energy was increased, the RC beams tend to reach ultimate state with sheet rupturing. To establish an appropriate strengthening method for existing impact resistant RC structures, it is important to solve AFRPs rupturing mechanism and develop a restraining method.

From these points of view, in this study, in order to investigate effects of the sheet volume on impact resistant capacity of RC beams strengthened with AFRPs, falling-weight impact tests were conducted taking unit mass of AFRPs and falling height of the weight as variables. In addition, Poly-Ethylene Telephthalate FRP sheet (PFRPs) was tried to use as absorbing material for restraining rupture of AFRPs, in which PFRPs was bonded combining with AFRPs. Because the elastic modulus of PFRP sheet is less than one tenth that of AFRPs, even though the ultimate strain is larger than four times that of AFRPs.

EXPERIMENTAL OVERVIEW
Specimens
Specimens used in this experiment were listed in Table 1. In this table, the nominal name of specimen was designated in order of strengthening procedure (N: case of without strengthening, A1/2/3: cases of bonding AFRPs with nominal tensile capacity of 600, 1,200, and 1,800 kN/m, respectively, AP: case of bonding AFRPs with 600 kN/m nominal tensile capacity combining PFRPs with 600 kN/m nominal tensile capacity) and setup falling height H of the weight. Here, the estimated free falling height Hr of weight was evaluated using measured falling velocity of the weight just before impacting the upper surface of the RC beam. In this experiment, the AFRPs bonded for the beams A1/2-H3.0 were finally ruptured, and those for the other beams were not ruptured but debonded partially.

### Table 1 List of specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Unit mass of sheet (g/m²)</th>
<th>Setup falling height of weight H (m)</th>
<th>Estimated free falling height of weight Hr (m)</th>
<th>Tensile stiffness of sheet (kN/m)</th>
<th>Tensile capacity of AFRPs (kN/m)</th>
<th>Failure mode of AFRPs*</th>
</tr>
</thead>
<tbody>
<tr>
<td>N-H2.5</td>
<td>AFRP: 415</td>
<td>2.5</td>
<td>2.43</td>
<td>-</td>
<td>-</td>
<td>PD</td>
</tr>
<tr>
<td>A1-H2.5</td>
<td>PFRP: 830</td>
<td>2.5</td>
<td>2.36</td>
<td>33.7</td>
<td>600</td>
<td>R</td>
</tr>
<tr>
<td>A1-H3.0</td>
<td>415</td>
<td>3.0</td>
<td>2.72</td>
<td>67.5</td>
<td>1,200</td>
<td>PD</td>
</tr>
<tr>
<td>A2-H2.5</td>
<td>1245</td>
<td>3.0</td>
<td>2.72</td>
<td>101.2</td>
<td>1,800</td>
<td>PD</td>
</tr>
<tr>
<td>A3-H2.5</td>
<td>3.5</td>
<td>2.64</td>
<td>2.72</td>
<td>2.72</td>
<td>1,200</td>
<td>PD</td>
</tr>
<tr>
<td>A3-H3.0</td>
<td>2.72</td>
<td>1,200</td>
<td>2.72</td>
<td>2.72</td>
<td>1,200</td>
<td>PD</td>
</tr>
<tr>
<td>A1-H3.5</td>
<td>3.5</td>
<td>2.72</td>
<td>2.72</td>
<td>2.72</td>
<td>1,200</td>
<td>PD</td>
</tr>
<tr>
<td>A2-H3.5</td>
<td>3.5</td>
<td>2.72</td>
<td>2.72</td>
<td>2.72</td>
<td>1,200</td>
<td>PD</td>
</tr>
<tr>
<td>A3-H3.5</td>
<td>3.5</td>
<td>2.72</td>
<td>2.72</td>
<td>2.72</td>
<td>1,200</td>
<td>PD</td>
</tr>
<tr>
<td>AP-H3.0</td>
<td>3.5</td>
<td>2.72</td>
<td>2.72</td>
<td>2.72</td>
<td>1,200</td>
<td>PD</td>
</tr>
<tr>
<td>AP-H3.5</td>
<td>1,250</td>
<td>3.10</td>
<td>2.72</td>
<td>42.8</td>
<td>1,200</td>
<td>PD</td>
</tr>
</tbody>
</table>

* R: Rupture, PD: Partial debonding

### Figure 1 Dimensions of beam, layouts of rebar and FRP sheet

![Figure 1 Dimensions of beam, layouts of rebar and FRP sheet](image)

### Table 2 Material properties and calculated load-carrying capacities of beam

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Compressive strength of concrete (MPa)</th>
<th>Yield strength of main rebar (MPa)</th>
<th>Calculated Flexural load-carrying capacity (kN)</th>
<th>Calculated shear load-carrying capacity (kN)</th>
<th>Shear-bending capacity ratio α</th>
</tr>
</thead>
<tbody>
<tr>
<td>N-H2.5</td>
<td>23.4</td>
<td>355</td>
<td>50.2</td>
<td>265.6</td>
<td>5.29</td>
</tr>
<tr>
<td>A1-H2.5/3.0</td>
<td>23.4</td>
<td>355</td>
<td>73.7</td>
<td>265.6</td>
<td>3.60</td>
</tr>
<tr>
<td>A2-H2.5/3.0</td>
<td>32.0</td>
<td>369</td>
<td>98.2</td>
<td>274.3</td>
<td>2.79</td>
</tr>
<tr>
<td>A3-H3.0/3.5</td>
<td>33.4</td>
<td>369</td>
<td>113.0</td>
<td>274.3</td>
<td>2.44</td>
</tr>
<tr>
<td>AP-H3.0/3.5</td>
<td>33.4</td>
<td>359</td>
<td>85.4</td>
<td>275.5</td>
<td>3.23</td>
</tr>
</tbody>
</table>

Figure 1 shows dimensions of the beam, layouts of rebar and FRP sheet. The beams have a rectangular cross section of 250 × 200 mm (height × width) and a clear span length of 3.0 m. The axial rebars of D19 (SD345) were placed at the upper and lower edges. These rebars were welded to steel plates placed at the both edges so as to reduce the anchoring length of rebars. Stirrups of D10 (SD295) were placed at the intervals of 100 mm. The FRPs was bonded onto the tension-side surface of the beams leaving 50 mm between the supporting point and the end of the sheet as shown in Figure 1. In the cases of beams A3/3AP, AFRPs with 1,200 kN/m or PFRPs with 600 kN/m tensile capacity were bonded as the first layer, respectively. And then, AFRPs with 600 kN/m tensile capacity was bonded as the second layer. Curing period for bonded FRPs was taken about 1 week under an atmosphere with temperature of 20 degrees Celsius.

Table 2 shows material test results for concrete and axial rebar, and calculated load-carrying capacity of each beam. The calculated load-carrying capacity of the beam was estimated following Japanese concrete standard (Japan...
Society of Civil Engineers, 2005) and using material properties of concrete and FRPs obtained from the test results, and nominal values for FRP sheets. Especially, flexural load-carrying capacities of the beams were estimated assuming perfect bonding between concrete surface and FRPs up to calculated ultimate state. Also, shear-bending capacity ratio was evaluated dividing calculated shear load-carrying capacity by flexural one. It is seen that these values for all specimens were greater than 1.0. This means that whole beams considered here will be statically reached ultimate state with a flexural failure mode.

![Experimental setup](image)

**Table 3 Material properties of FRPs**

<table>
<thead>
<tr>
<th>FRPs</th>
<th>Unit mass (g/m²)</th>
<th>Nominal tensile capacity (kN/m)</th>
<th>Thickness (mm)</th>
<th>Tensile strength (GPa)</th>
<th>Elastic modulus (GPa)</th>
<th>Ultimate elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AFRPs</td>
<td>415</td>
<td>600</td>
<td>0.286</td>
<td>2.10</td>
<td>118</td>
<td>1.8</td>
</tr>
<tr>
<td>PFRPs</td>
<td>1,250</td>
<td>600</td>
<td>0.906</td>
<td>0.74</td>
<td>10</td>
<td>7.0</td>
</tr>
</tbody>
</table>

**Test procedure**

Experimental setup is shown in Figure 2. Falling-weight impact tests were conducted following a single loading method, in which a falling-weight was dropped just one time from the predetermined height. Mass and diameter of the weight were 300 kg and 200 mm, respectively. Loading point was limited at the mid-span of the beam. Also, each beam was pinned supported and was pinched on its top and bottom surfaces at the lines of 200 mm inside from the both edges. The impact and reaction forces were measured using load-cells for impact loading. Based on our previous research works, ultimate state of the beam was defined as follows: 1) the residual deflection was reached up to 2% of the span length; and/or 2) FRP sheet was completely debonded and/or ruptured.

Table 3 shows a list of material properties of FRP ses used in this experiment. Here, it is seen that an elastic modulus of PFRPs is less than one tenth that of AFRPs, but the ultimate strain is larger than four times that of AFRPs. Measuring items were the impact force $P$, the reaction force $R$, the mid-span deflection $\delta$, and the axial strains of AFRPs. These output data were continuously recorded by using digital data recorders with a sampling time of 0.1 ms/word. Also, failure behaviour around the loading point was recorded by using a digital high-speed camera with the intervals of 1 ms/frame.

**EXPERIMENTAL RESULTS AND DISCUSSIONS**

**Time histories of impact force, reaction force, and deflection**

Figure 3 shows time histories of the impact force $P$, the reaction force $R$, and the deflection $\delta$ for all specimens. From Figure 3(a), it is observed that the time history of the impact force $P$ consists of a predominant first half-sine shape with about 3 ms duration time and subsequent high-frequency components, irrespective of sheet volume, sheet material, and falling height $H$ of the weight. The maximum impact forces for strengthened RC beams were larger than that for not strengthened one.

From Figure 3(b), it is seen that configurations of the time history of the reaction force $R$ for all beams were composed of two components: a sinusoidal half shape with about 35 to 50 ms duration time; and a high frequency component with a period of about 5 ms. In the case of falling height of the weight $H = 2.5$ m, duration times of
main component for the strengthened beams tend to be smaller than that for the beam N. This means that the flexural rigidity of the strengthened RC beams can be upgraded due to bonding AFRPs. In the case of \( H = 3.0 \) m, duration times of the beams A3/AP were shorter than those for the beams A1/A2. This means that the flexural rigidity of strengthened beams was increased corresponding to an increase of sheet volume and a rupture of the sheet could be restrained due to increasing sheet volume or combining with PFRPs. Also, in the cases of \( H = 3.0 \) and 3.5 m, comparing the duration time between beams A3 with AP, it is seen that the case of beam AP is slightly larger than the case of beam A3.

From Figure 3(c), it is observed that configurations of the time history of the deflection \( \delta \) for all cases show a damped sinusoidal shape after unloaded and the maximum/residual deflections tend to be restrained corresponding to an increment of the sheet volume. In the case of \( H = 2.5 \) m, it is seen that as maximum deflections of the beams A1/2 were smaller than that of beam N, the deflection can be reduced due to bonding FRPs. In the case of \( H = 3.0 \) m, it is seen that the deflections of the beams A1/2 were larger than those of the beams A3/AP, because the beams A1/2 reached the ultimate state with sheet rupturing. Also, in the cases of \( H = 3.0 \) and 3.5 m, comparing configurations of the time history of the deflection between beams A3 and AP, maximum value and duration time of the time history for beam AP were greater than those of beam A3. This means that the flexural rigidity of the beam AP may be less than that of the beam A3. Also, the bonded AFRPs in the case of the beam AP-H3.5 was partially ruptured as shown in Fig 4.

From these results, it is confirmed that the deflection of the RC beams subjected to impact load can be restrained by bonding FRPs. Also, the effect was increased corresponding to an increment of the sheet volume and/or combining PFRPs.
Transitions of strain distribution of FRP sheet and failure behaviour of RC beams

Figures 5 show transitions of the strain distribution of AFRPs and failure behaviour of the beams around the loading point for the beams A2/A3/AP-H3.0 at the elapsed times from 1 to 20 ms from the beginning of the impact. From these figures, it is observed that until the elapsed time of \( t = 4 \) ms, flexural and diagonal cracks were developed, and strains of the AFRPs were gradually increased around the loading point for all beams. At the elapsed time of \( t = 8 \) ms, these cracks were widely opened, and the strains of the AFRPs were distributed in the whole span. After the elapsed time of \( t = 9.5 \) ms, in the case of the beam A2-H3.0, the upper concrete cover has been crushed, and the strain at the mid-span reached about 1.5\%. On the other hand, in the case of the beams A3/AP-H3.0 at the elapsed time of \( t = 12.0 \) ms, the upper concrete cover has been crushed similarly to the case of the beam A2-H3.0, and also AFRPs was partially debonded due to peeling action of the critical diagonal crack developed in the lower concrete cover near the loading point. Maximum strain was about 1.25\% and not concentrated but distributed in the region of around the loading point.

Finally, in the case of the beam A2-H3.0 at the elapsed time of \( t = 10.0 \) ms, many cracks were observed in the lower concrete cover, and then AFRPs was ruptured around the loading point. On the other hand, in the cases of the beams A3/AP-H3.0 at the elapsed time of \( t = 20.0 \) ms, the sheet debonded area and large strain developed area were widely spread toward both supporting points.

From these results, it is confirmed that the strain concentration and rupturing of sheet can be restrained by increasing sheet volume and/or combining with PFRPs.
CONCLUSIONS

To investigate effect of the sheet volume on impact resistant behaviour of the RC beams strengthened with bonding AFRPs, falling-weight impact loading was conducted taking unit mass of AFRPs and falling height of the weight as variables. In addition, PFRPs was also applied combining with AFRPs to absorb the impact force and/or to restrain rupturing of the AFRPs. From these experiments, following results were obtained:

1) The mid-span deflection of the RC beams can be reduced due to bonding AFRPs in flexure;
2) The impact resistant capacity of the beams will be increased corresponding to an increase of the AFRPs volume; and
3) Axial strain concentration of the AFRPs and rupturing of the sheet can be restrained by increasing of sheet volume and/or applying PFRP sheet in combination with AFRPs.

Figure 6 Transitions of strain distribution of FRPs and failure behaviour of RC beam (continued)

ACKNOWLEDGMENTS

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DURABILITY OF DRY AND IMPREGNATED REINFORCEMENT FIBRES EXPOSED TO DIFFERENT ALKALINE ENVIRONMENTS

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ABSTRACT

The durability of fibrous composite materials used as strengthening systems in civil engineering is of primary importance in order to calibrate design guidelines and to assess the design life of the strengthened structure. Recent applications of Fibre Reinforced Mortar (FRM) systems, used as external strengthening in masonry and concrete buildings, have shown that dry fibres may be used as long reinforcement within a cementitious or lime-based mortar. This situation is quite different from FRP (Fiber Reinforced Polymers) composites in which, typically, an epoxy resin acts as a protection for the fibres, which are not in direct contact with the pore solutions of the substrate. In this study an extensive campaign was conducted with the aim of evaluating possible methods for the assessment of the durability of fibrous reinforcement in alkaline environment.

Two different configuration of reinforcement were studied: dry fibrous reinforcement and industrially pre-cured fibrous reinforcement (with different polymer resins). This choice was done since at the moment different reinforcement systems are used in FRM systems, which can be in forms of dry fibres or FRP grids. Glass fibres having different chemical composition (E-glass, AR-glass), basalt fibres, steel fibres, aramid fibres and carbon fibres were tested and used as benchmark. Four different alkaline environments were chosen for the accelerated exposure, according to the existing technical recommendation and scientific literature. In this perspective the presence of different ions and pH levels were accounted. Different exposure times were considered, from 168 up to 4320 hours; the wet environments were maintained at different temperatures in order to see possible acceleration effects. The comparison was made in terms of residual tensile strength, since all unexposed and conditioned specimens were tested up to failure under uniaxial tensile forces. SEM microscopy images are presented to show the damage at fibre level which was observed in vulnerable fibres.

KEYWORDS

Durability, alkaline, composites, fibres, glass, aramid, basalt, carbon, steel.

INTRODUCTION

In the last years new inorganic matrices were used, in place of epoxy resin. They are made of lime mortar, cement mortar or polymer added cement/lime mortar. Fibrous reinforcement systems which are immersed into this environment need to be studied in terms of durability due to the high alkalinity of the interstitials pore solutions. In fact pH values, in interstitial solution present within the pores of the young cementitious matrix, may be comprised between 12.5 and 13.5, depending on the chemical species which are present. The reinforcing types most commonly used in this field are constituted by: fabrics, soft meshes or nets (glass, carbon, polyaramid, natural fibres) which are embedded directly into the inorganic binder (cement or lime). Preformed FRP grids, plates or bars, commonly constituted by glass or carbon fibres and thermosetting resins are used as alternative solutions. As regards the exposure in an alkaline environment, object of this study, it is crucial to identify the possible vulnerability of reinforcement systems, especially those more sensitive made with glass or basalt fibres. In these cases the chemical sensitivity of the material forming the reinforcement, could lead to a chemical damage, even severe, of the reinforcement itself, with consequent reduction of the mechanical properties at macroscopic level. To assess and measure this potential sensitivity, laboratory tests were conducted by immersing dry and cured fibrous materials in aqueous solutions with pH between 12 and 13.5, by varying the exposure time or speeding up the diffusive effect of such solutions through a increase of temperature of the alkaline bath. In order to simulate several possible service conditions, alkaline solutions may have different chemical composition, containing varying percentages by weight of alkali hydroxides such as Na(OH), K(OH), Ca(OH)₂. This choice was done by considering previous literature that testifies that for the glass and basalt fibres different degradation mechanisms
have been identified (Gao et al. 2003, Purnell and Beddows 2005, Butler et al. 2010,) that can be summarized in the following cases: massive corrosion of the filaments due to attack by OH- ions, rupture for static fatigue due to growth of existing surface defects aggravated by OH- ions in the vicinity of the edges of the defects, densification due to the precipitation of crystals, which go to fill the spaces between the filaments of the fiber with subsequent embrittlement. The glass fibres normally used in the composite industry (E-glass) have demonstrated a remarkable sensitivity to alkaline attack, resulting in poor durability. The mechanisms of chemical attack in an alkaline environment are classifiable in two modes: leaching due to diffusion of alkaline ions that cause a dissolution of the glass fibres; etching due to breaking of the bond Si-O-Si, which forms the main structure of the glass fiber, because of the attack lead by the alkaline chemical species. When the fibres are used in a composite polymer matrix, the protective role of resin is important too, which should not degrade in order to maintain its mechanical role. As regards the resins the most dangerous of degradation mechanism is hydrolysis of the ester groups present within the polyester resins and vinyl ester (Chen et al. 2007). The vinyl ester resins have a much lower concentration of ester groups and thus a reduced sensitivity to hydrolysis compared to conventional polyesters. Epoxy resins have no ester groups and therefore are much less sensitive to hydrolysis, thus they are commonly used in structural composites. Furthermore, the durability of FRP products made from E glass fibres, in an alkaline environment, is reduced compared to the use of AR glass fibres, and this may change also depending on the resin used for the impregnation. Laboratory tests are often conducted with the aim of accelerating the effects, in order to simulate long exposure times in situ. This is done by increasing the temperature of the alkaline bath. This change in test temperature strongly affects on durability, and various studies show that accelerated aging tests (T=50-60 °C) seem to enhance the degradation processes of Glass FRP (GFRP), as found by Bennokrane et al. (2001) and Micelli and Nanni (2004).

In the present paper the results of an extensive experimental campaign are presented. Dry reinforcing fibres (glass, carbon, basalt, polyaramid, steel) and impregnated fibres (glass and carbon), were subjected to tensile tests after accelerate aging in alkaline bath, following different ageing protocols. The values of the residual tensile strength are compared; local damages at fibre level will be also shown through SEM electronic microscopy.

EXPERIMENTAL PROGRAM

For the evaluation of the durability of the fibres in alkaline environments it was decided to choose the experimental variables based on the tests already carried out in the literature and according to existing guidelines in the civil engineering field. The residual tensile strength of the fibres was chosen as reference parameter to compare possible effects of mechanical degradation, even if an electronic microscopy was also used to observe the surface of the fibres after conditioning. This choice was followed since it is well known that durability in terms of structural design is the ability of the tested materials to maintain the mechanical properties after a certain period of exposure in a certain aggressive environment.

All samples of the same material were extracted from the same coil, in order to make meaningful comparisons, and purify possible systematic errors due to differences in production batches. Thus the fiber content (typically expressed in TEX unit g/km) for each type of material is identical in each sample. Due to the possible error in determining the real cross section of the specimens, residual capacity is expressed in terms of ultimate tensile load (kN). For the execution of the tensile tests recommendations of the CNR DT 203 (CNR 2006) were used for the preparation of the samples and the UNI EN ISO 527/4 with regard to the mechanical tests. The minimum number of repetitions for each type of sample was chosen equal to 5, in order to make the experimental the results statistically significant.

Four alkaline environments were used for the long-term simulated ageing of the fibres. The first wanted to simulate immersion in a lime mortar, the second the presence of a cementitious mortar, and were taken from existing literature. The other two environments were not directly related to the application of structural strengthening systems in civil structures, but were studied in order to detect the sensitivity according to the available standard protocols provided by ETAG and ASTM. The following formulations were used, with content of chemical species expressed in weight %:

- Environment A - lime mortar (Waldron et al. 2001): 0.16% Ca(OH)_2 - pH =12.6
- Environment B - cement (Micelli and Nanni 2004): 0.16% Ca(OH)_2, 1% Na(OH), 1.4% K(OH) - pH = 13
- Environment C - ETAG 029 / Annex A (2013) : 0.2% K(OH) - pH = 12.5
- Environment D - ASTM E 2098 (2006) :5% Na(OH) - pH = 14

The experimental protocol is evidenced in Table 1, in which exposure times, temperatures and conditions are summarized for the dry fibres.
For the dry five specimens, per each exposure/test protocol and per each material were prepared. Dry fibre bundles were cut with a length of 580 mm, after the conditioning period the ends of the dry fibre specimens were impregnated with epoxy resin for a length of 100 mm in order to better grip them inside the universal testing machine.

Table 1 Exposure to alkaline environments (dry fibres)

<table>
<thead>
<tr>
<th>Environment</th>
<th>Temperature (°C)</th>
<th>Exposure times (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>20 ± 3</td>
<td>7, 30, 90, 180</td>
</tr>
<tr>
<td>A</td>
<td>45 ± 5</td>
<td>7, 30, 60</td>
</tr>
<tr>
<td>B</td>
<td>20 ± 3</td>
<td>7, 30, 90, 180</td>
</tr>
<tr>
<td>B</td>
<td>45 ± 5</td>
<td>7, 30, 60</td>
</tr>
<tr>
<td>C</td>
<td>20 ± 3</td>
<td>7, 30, 90</td>
</tr>
<tr>
<td>D</td>
<td>20 ± 3</td>
<td>7, 30</td>
</tr>
</tbody>
</table>

During the conditioning the pH level was measured daily, and the following changes were recorded: environment A - lime mortar T=20°C: pH = 12.6 ± 0.2 for the first 30 days, then a reduction until pH= 12.0 ± 0.2 was recorded after 90 days; environment A - lime mortar T=45°C: pH = 12.6 ± 0.2 constant after 60 days; environment B - cement pH = 12.8 ± 0.2 constant after 60 days in all cases; environment C - ETAG 029 / Annex A : pH = 12.5 ± 0.2 for the first 30 days, then a reduction until pH= 11.0 ± 0.2 was recorded after 90 days; environment D - ASTM E 2098: pH = 13.5 constant after 30 days. The following dry fibres were conditioned in alkaline bath:
- Glass Fibres E - TEX: 4800 g/km
- Glass Fibres AR 1 - TEX: 2400 g/km (Alkali resistant – presence of ZrO₂)
- Glass Fibres AR 2 - TEX: 2400 g/km (Alkali resistant – presence of ZrO₂)
- Carbon Fibres - TEX: 1600 g/km
- Basalt Fibres 1 – extracted from a dry net with a grammage of 200 g/m²
- Basalt Fibres 2 – extracted from a dry net with a grammage of 200 g/m²
- Polymaramidic Fibres (PBO) - extracted from a dry net with a grammage of 200 g/m²
- Steel strand - extracted from a dry net with a grammage of 670 g/m²

Glass fibres AR-1 and AR-2 differ for the chemical composition, having different content of ZrO₂. E-glass and Basalt fibres 1 and 2 are related to different manufacturers. PBO fibers are the only available for structural strengthening of civil and heritage building; steel strands were also taken from a commercial product that is used for structural strengthening of concrete and masonry structures.

As it regards pre-cured impregnated strands, they were cut from FRP grids produced through an industrial process that controls the curing of the polymeric resins. Three different meshes were tested:
- Mesh with 50% E glass + 50% AR glass fibres in a vinylester -epoxy matrix
- Mesh with 100% AR glass fibres in a vinylester -epoxy matrix
- Mesh with 100% carbon fibres in a bisfenolic matrix

All meshes were produced by using strands of 9600 g/km, and the fiber content (% mass) inside the composite strands was 67%. For impregnated fibers ten specimens per each exposure/test protocol and per each material were prepared and tested. The ageing program for pre-cured FRP strands is illustrated in Table 2 of the following section, which will report also the experimental results by comparison of mechanical tests.

EXPERIMENTAL RESULTS

Dry fibres

The results of the tensile strength performed on control specimens (unconditioned) highlighted a coefficient of variation (COV%) between 3% and 12%, thus it should be considered the range of the experimental error. In Figures 1 and 2 the residual tensile strength is plotted, at different exposure periods, for the different dry fibrous reinforcements that were tested after ageing.
Form the results in Figure 1 it is clear that different vulnerability was found for the different materials and for the different environments, as expected from the experience in the field. E glass fibers and basalt fibers demonstrated to be highly sensitive to the attack of alkaline ions, while the protection offered by the ZrO$_2$ in AR glass fibres strongly mitigated the chemical attack, on both AR-1 and AR-2. This is clearly evident in the environments A and B which are of major interest in the field of FRM-strengthened construction. In the case of lime mortar bath (environment A) the tensile strength of E-glass and basalt fibres had the same maximum drop of more than 30%. In the case of cement solution (environment B) the tensile strength of E-glass had a maximum drop of 76%, while for the case of basalt it was 35%. Environment C confirmed the same trend, even if exposure time was shorter. The case of environment D demonstrated to be very aggressive for all the fibres except for steel and carbon fibres which remained totally unharmed. Even if it is known that concentrations of alkaline ions as same as those used are not realistic, it is considered a valid “stress test” to compare the different susceptibility of the different chemical composition of the fibres.

![Figure 1](image1.png)

**Figure 1** Strength retention after alkaline exposure of dry fibers at T=20°C

The results plotted in Figure 2 are referred to tests after accelerated ageing by increasing the temperature at 40°C during the exposure. The effects of the temperature were very strong since all fibres, except carbon (steel was not tested due to its insensitivity) had a significant reduction of final strength, even if E glass and basalt fibres remained those much more vulnerable respect to AR glass and PBO.

In both cases of ageing at low and high temperature the cementitious simulated environment demonstrated to be more aggressive respect to lime simulated bath. This is due to the presence of different ions, which act as responsible of the damage growth on the surface of the fibres. It is known, in fact, that environments that contain in calcium ions would seem that, after the attach of these calcium ions to the Si-O- group that is formed (chemical degradation mechanism), a surface layer that acts as a barrier to the diffusion of ions OH- is formed. This occurrence delays the process of damage growth. This beneficial effect was not seen in alkaline environments in which there was the presence of sodium or potassium ions.

The SEM microscopy images of Figures 3 show the damage in E glass fibres produced by the attack of the alkaline ions with reference to the environment A at 40°C after 30 days. Figure 4 illustrates how this damage is not present on AR-1 glass fibres; this is due to the presence of zirconium oxide which has commonly a weight content between 16% and 20%. In these cases, during the exposure in an alkaline bath, a layer of oxide is present on the surface of
the fibres, which greatly increases the chemical resistance against alkaline attack. This chemical barrier allows to slow down very significantly the spread of the hydroxyl OH- that would cause the breakage of the glass filaments.

**Figure 2** Strength retention after alkaline exposure of dry fibers at T=45°C

**Figure 3** SEM Microscopy: E-glass dry fibres (A environment)

**Figure 4** SEM Microscopy: AR1-glass dry fibres (A environment)

**Pre-cured impregnated fibres**

The results of the durability tests on pre-cured specimens allowed to strongly highlight the important role of the matrix as protective binder in the composite structure. It was expected that when the matrix is not penetrated by the diffusion of alkaline solutions, fibres damage should be less severe than the cases of dry fibres. This evidence was found also in this experimental study, even if different fibres and different resin composition were used in order to test possible solutions used in the real field. In these cases, in addition to hydrolysis, the deterioration of the matrix could be present due to phenomena of plasticization and / or swelling caused by the diffusion of the alkali solution within the resin itself. This could lead to a reduction of the capacity of the resin to transfer the loads between the fibres which translates into a mechanical decay of the entire composite. The alkaline attack may also
take place at the fibre-resin interface, and also in this case the macro-mechanical properties of the composite would strongly decrease. The permeability of the resin typically depends from the type of resin (macromolecular composition) and the degree of crosslinking. In this study the resins that were used were cured through a controlled industrial program at temperatures that go from 100 °C to 120 °C, which means that the glass transition temperature and degree of crosslinking are remarkably higher than those cases in which the resin is cured on site. The results in Table 2 show that the loss in tensile strength is negligible, except for the case of environment B after 40 days, in which the strength had a drop of 28%.

Table 2 Ageing protocol and residual tensile strength for pre-cured FRP strands

<table>
<thead>
<tr>
<th>Fibre</th>
<th>Environment</th>
<th>Temperature (°C)</th>
<th>Exposure times (days)</th>
<th>Residual Tensile Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>50% E glass-50% AR glass</td>
<td>A</td>
<td>40 ± 3</td>
<td>30</td>
<td>0.94</td>
</tr>
<tr>
<td>Vinylester-epoxy matrix 50% E glass-50% AR glass</td>
<td>B</td>
<td>40 ± 3</td>
<td>40</td>
<td>0.72</td>
</tr>
<tr>
<td>Vinylester-epoxy matrix 50% E glass-50% AR glass</td>
<td>C</td>
<td>40 ± 3</td>
<td>7</td>
<td>0.92</td>
</tr>
<tr>
<td>Vinylester-epoxy matrix 100% AR glass</td>
<td>A</td>
<td>40 ± 3</td>
<td>30</td>
<td>0.89</td>
</tr>
<tr>
<td>Vinylester-epoxy matrix 100% carbon</td>
<td>A</td>
<td>40 ± 3</td>
<td>30</td>
<td>0.98</td>
</tr>
</tbody>
</table>

CONCLUSIONS

A study on the durability of fibrous reinforcements was presented. Dry fibres strands and pre-cured strands were subjected to ageing in different alkaline environments. E-glass and basalt fibres demonstrated to be more sensitive and vulnerable respect to the other fibres type. AR glass confirmed their ability to resist also in lime or cement simulated alkaline baths. The presence of the vinylester-epoxy resin in pre-cured glass fibres specimens allowed to strongly reduce the effects of the alkaline attack in lime simulated environment.

ACKNOWLEDGMENTS

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Field Applications
A REVIEW OF GEOTECHNICAL APPLICATION OF FIBER REINFORCED POLYMER MATERIALS

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ABSTRACT

Fibre Reinforced Polymer (FRP) has been widely used for reinforcement or rehabilitation to the upper structure. Currently, some research has investigated the application of FRP underground structure which refers to reinforcement of the structure or substitution of traditional material. Due to the corrosion of steel bar or deterioration of concrete in harsh environments, there is a need to replace some steel reinforced concrete piles which leads to the study of composite piles including FRP piles. However, not much has been done to examine the application of FRP in geotechnical structures. This paper presents an overview of FRP in geotechnical application with focus on FRP reinforced piles. Furthermore, the geotechnical utilisation in shield construction and further investigation of the FRP pile are highlighted. It is expected that this review could provide practical information for geotechnical engineers.

KEYWORDS

FRP bar, FRP slice, reinforcement, geotechnical application, shield construction.

INTRODUCTION

Fibre Reinforced Polymer (FRP) is a composite material which contains fibre that can provide high tensile strength and resin. This material is also known as fibre-reinforced plastics, or advanced composite materials (ACMs) (Bank & Wiley, 2006). FRP is well known for its high ratio of strength to weight, high ratio of longitudinal/transversal Young’s and high ratio of longitudinal/transversal shear modulus (Pecce, 2001). It can be categorized into Aramid-FRP (AFRP), Basalt-FRP (BFRP), Carbon-FRP (CFRP) and Glass-FRP (GFRP) depending on the materials of fibre, and due to this, the FRP product is more flexible to change the material properties via designing volume ratio of fibres to resin and selecting the types and orientation of fibre. It has been widely applied in aerospace, automotive, industrial and recreational products before the application in civil engineering construction. The FRP composite products in the market are mostly manufactured into FRP sheets and FRP bars for construction purposes. FRP bars usually include GFRP anchor (ribs) and GFRP grille (Zhang et al. 2001). There is a large amount of research focus on the analysing the structural behaviours of beams, walls with consideration of FRP sheet application (Mostofinejad & Mohammadi Anaei, 2012; Mohammed et al. 2013; Mostofinejad & Tabatabaei Kashani, 2013; Mosallam et al. 2015). A notable method utilising the confinement to concrete columns by FRP has been studied and the results show that methodology could enhance strength when the structure suffers axial loading (Kwan et al. 2015; Lo et al. 2015; Youssf et al. 2015). The application history of the FRP composite materials has surpassed 80 years. However, these pass projects mostly refer to the rehabilitation or strengthening of the structure members such as beams and columns. This paper presents an overview of FRP application in geotechnical works.

DEVELOPMENT AND INVESTIGATION OF FRP PILE

There are wide utilisations of piles whereby some issues occur when piles are located in harsh environments especially in marine or coastal conditions. The piles with traditional materials could be destroyed due to corrosion of steel, deterioration of timber and degradation of concrete as shown in Figure 1. The deterioration of the timber, concrete, and steel piling systems costs the United States nearly 2 billion dollars per year for repair and replacement (Hassan & Iskander, 1998). In 1998, the Federal Highway Administration (FHWA) initiated the project on the use of the Fibre-reinforced Polymer pilings as the replacement of traditional piles under the background of a waterfront rehabilitation project which one goal was the replacement of up to 100,000 bearing piles for the lightweight structures.
The development of concrete piles has a long history. Engineers are now facing the problem relating to piles, that even though the piles are made of concrete and steel which possess good rigidity and high strength, damage still occurs as shown in Figures 2 and 3 which depict the damage of concrete piles photographed in Gold Coast and Adelaide in Australia, respectively.

The application history of the FRP piles is approximately three decades. Dated back to April 1987, the first prototype recycled pile was driven at The Port of Los Angeles (Juran & Komornik, 2007). This replacement of creosote-treated timber piles successfully avoided the threat of marine borers. As early as 1998, the Empire State Development Corporation (ESDC) undertook a waterfront rehabilitation project known as Hudson River Park. That project was involved replacing up to 100,000 bearing piles for lightweight structures (Juran & Komornik, 2007), the concrete-filled FRP composite piles then were employed by Virginia Department of Transportation (VDOT) in 2000 for an entire bent of the new Route 40 Bridge over the Nottoway River in Sussex County, Virginia (Pando, 2003a; Pando et al. 2004).
Even though this material is very expensive, the overall cost in the long run is more economical than the traditional materials of concrete and steel, due to advantages of low management cost and long service life. Ballinger pointed out that although the cost of FRP composite materials may be higher, the cost of labour and use of equipment necessary for construction work may be lower due to their lighter weight (Pando, 2003b). Aside from the cost, there is a growing concern in the environmental and health impact of using treated timber and steel painted with solvent or heavy-metals coatings (Guades et al. 2012). These poisonous materials threatened the marine borers which lead to the engineers starting to replace these timber piles with FRP piles which are made of fibre and plastics or resins. Robinson and Iskander (2008) pointed out those using recycled plastics to manufacture FRP piles utilises material which may have been otherwise landfill and can be more economical in aggressive environments when life-cycle costs are considered. According to the data from EPA (2006) that less than 10% of the 13.7 million tons (equal to loading of 122GN) of plastic containers and packaging produced annually in the U.S. are recovered by recycling (Brent & Iskander, 2008).

Many researchers have started to examine the FRP piles. Most of the researchers paid attention to: the drivability, driving efficiency, durability, and surface friction between FRP and soils; effect of the hammer; and resistance of the soil. The research of the driving hammer effect was conducted using wave equation analysis which considered the effects of weight and velocity of the hammer and pile property. This showed that the single-acting steam hammer was more efficient than the diesel hammer as it could drive the composite deeper with the same number of blows (Iskander & Hassan, 1998). A modelling study simulated via the software program Microwave, showed that both hollow and concrete-filled FRP piles could be driven by a heavier hammer (Ashford & Jakrapiyanun, 2001). Moreover, the soil resistance to driving which included side friction and end bearing resistance was also been investigated using the wave equation analysis program (WEAP), and the results from the entire spectrum of the study demonstrated that, for the drivability, there was more substantial difference in friction and end bearing conditions for concrete-filled FRP and concrete piles (Mirmiran & Shahawy, 1996). The outcome of a study which researched the interface behaviour between sand and FRP (Fam, 2000) concluded that FRP exhibits similar relationships between peak interface friction coefficients and the relative roughness for a given granular material (Pando et al. 2002).

A review on the driving performance of FRP composite piles pointed out that the types of composite piles are Steel Pipe Core Piles, Structurally Reinforced Plastic Piles (SRP), Concrete-filled FRP Piles, Fiberglass Pultruded Piles, Fiberglass Reinforced Plastic Piles, Hollow FRP Piles and FRP sheet Piles as shown in Figure 4 (Guades et al., 2012). A further study of Pando (2003b) emphasized that Composite products are available in the market, which are steel pipe core piles, structurally reinforced plastic matrix piles, concrete-filled FRP pipe piles, fiberglass pultruded piles, plastic lumber piles, hollow FRP piles and FRP sheet piles.

Due to these results obtained from investigation, notable research of behaviours of FRP composite piles under vertical loads was conducted to determine the stiffness, flexibility, settlement and bearing capacities. Through this research, in-situ static and dynamic tests were conducted with onsite Standard Penetration Tests (SPT) and Cone Penetration Tests (CPT). 4 types of FRP piles static and dynamic load tests were conducted in Elizabeth, New Jersey which were concrete filled fiberglass shell piles (Lancaster Pile, Figure 5), polyethylene piles reinforced...
with steel bars (PPI Pile, Figure 6), polyethylene piles reinforced by fiberglass bars (SEAPILES, Figure 7) and solid polyethylene piles (American Ecoboard Pile, Figure 8). The tests have shown possible applicability of plastic piles to traditional axial loading applications and the further work of long term creep performance and durability of these piles was highlighted (Robinson & Iskander, 2008). Concrete-filled FRP piles came into notice and were considered the best FRP piles to resist upper loading. From a different perspective, the other types of FRP piles showed inadequate capacity, for example, the steel pipe pile is mostly used under the conditions of being exposed to water, which means the FRP is actually only used for protection away from corrosion. Further example like SRP, is mainly used in fendering applications and regarded as potential load-bearing piles (Guades et al., 2012). FHWA proposed the conclusion that the FRP composite piles can be used effectively as vertical load-bearing piles and represent an alternative for deep foundation construction, especially in waterfront environments and aggressive soils (Juran and Komornik, 2007).

Additionally, many researchers started to investigate the FRP composite piles that suffered from lateral loading. The comparison tests between FRP tube pile filled with concrete (no steel reinforcement) and prestressed concrete pile were conducted and the results showed that, after the statnamic test, the FRP tube pile responses was much less stiff than the prestressed concrete pile response (Pando & Filz, 2004). A similar research agreed that the concrete filled GFRP tube pile was more flexible, which depicted a larger displacement at equivalent lateral loads and led to the reduced service load capacity of the GFRP pile (Weaver et al. 2008), and it was also concluded that the concrete filled GFRP piles could be adequately modelled using traditional p-y curves and classical beam theory. The displacement behaviour of high strength glass fibre reinforced composite bored piles was researched and results were obtained from inclinometer and survey measurements, it was concluded that those two measurement methods were the same. Furthermore, the measured displacement was much higher than the pre-load test calculations and the post-load test analysis indicated an 85% reduction in soil strength was needed for predicting the deflection of the FRP pile (Thomann et al. 2004). The limitation of this tests equipment which is used for
providing the lateral loading is mostly applied forces from pile head without considering distributed loading along the pile length.

THE GEOTECHNICAL APPLICATION OF FRP BARS

FRP anchorage in slope reinforcement
A large amount of research focused on the investigation of reinforced concrete (RC) structures with externally bonded FRP application for the purpose of strengthening and repairing. One of the limitations of this technology though is the propensity of the FRP to prematurely de-bond at strains well below its rupture strain (Zhang et al. 2012). So some researchers conducted tests to design the FRP anchors and provided the criterion for optimal assessment. Other perspectives of the anchorage are the application of FRP bolt or FRP anchor bolts which are used for slope treatment in expansive soils. The reason for using FRP bolt is that traditional main governance methods of expansive soil slope is to use steel anchor bolt and frame beam or grid, but they are easily corroded and the durability is poor (Liu, 2014). Through comparison between the FRP anchor bolt and steel bolt and analysis, the author pointed out that even though the maximum crack widths were different, and the pulling resistances were almost equal.

Others application projects used for slope reinforcement is reported by Luo, where the technology of GFRP reinforcement anchor was adopted in the project of Changji Expressway built for the purpose of red sandstone slope reinforcement during construction in 2008. This proposed reinforcement system used GFRP bolts with 28mm diameter and the results demonstrated the slope was overall stabilized (Luo, 2014). Luo also provided another GFRP anchor application used in an underground retaining project in which GFRP bolts replaced the Chinese traditional HPB325 bolts and the result was successful after the pull-out tests.

FRP bars in shield construction
A Tunnel boring machine (TBM) is one of the most efficient equipment used for tunnelling, however TBM cannot easily break the walls or piles which are used for retaining the earth pressure. As shown in Figure 9, 2 yellow coloured blocks are the areas of the train entrances of the subway station, where the steel reinforced retaining structures are located. The old way to solve that problem is utilizing manual work which is dangerous to the workers and time consuming. The project of Yuantong subway station in Nanjing city suffered a human loss during breaking the retaining structures in 2007, where the soil layers collapsed and. In this accident, the ground cracked and extended to 150 meters (Liu et al. 2014).

The application of GFRP bars are the most commonly used reinforcement during shield tunnelling construction. The replacement of traditional steel reinforcement with GFRP reinforcement in concrete structures (pile or retaining wall) successfully solves the problems mentioned above. It is because GFRP shows brittle behaviour when loading reaches and surpasses failure conditions, as the TBM can directly cut the retaining structures. This technique is widely accepted in China and there are numerous projects that have applied GFRP to fully or partially replacing the traditional reinforcement. Some projects with GFRP geotechnical application (retaining wall) are Shanmei subway station, R2 in Dongwan city (Ming, 2011), Shenzhen Subway, R5 in Shenzhen city (Zhang et al,
FURTHER RESEARCH TO BE INVESTIGATED

Some investigations of the FRP composite piles that suffered with axial loading were conducted. However, the FRP slice confined piles lack research due to research only being focused on the FRP tube or FRP inside reinforcement. Therefore, further research considering geotechnical application of FRP laminar and investigating of the laminar layers and orientation of fibre is required.

In practice, laterally loaded piles can be classified as active piles or passive piles, with regard to the loading transferring direction between the piles and the surrounding soils (Beer, 1977). To the best of my knowledge, there is no research considering the passive FRP composite piles suffering soil movement. Furthermore, there are no proposed equations for determining the capacity of FRP piles which consider the fibre orientation and FRP reinforcement ratio.

As cases mentioned in shield construction, the GFRP bar reinforced concrete passive piles successfully replaced the traditionally reinforced concrete passive piles in deep excavation construction and create convenient condition for the TBM. However, there has been no deflection and stress investigation. Moreover, there is a limited data analysis associated with FRP laboratory tests.

Generally, the FRP composite piles are still experience brief time spans and the data obtained from onsite field tests are limited. More data needs to be recorded for the engineers in the future.

CONCLUSIONS

This paper has presented an overview of the FRP geotechnical application followed by a brief historical development of FRP material. Some of the projects are illustrated and the future research required is presented. The limitation of the vertical loaded FRP piles is that, there is no test conducted for the piles on FRP laminar application and there is no consideration of the layers and fibre orientation. For the lateral loaded pile, tests were mostly applied loading from the pile head and there was no research of the passive piles suffering from soil movement. Furthermore, the FRP bars application technology successfully replaced the conventional steel material in shield construction. However, there is still a lack in research and data analysis.

REFERENCES


A REVIEW ON DEVELOPMENT AND APPLICATIONS OF HYBRID COMPOSITE PLATE (HCP): A ROBUST RETROFITTING SOLUTION FOR RC MEMBERS

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ABSTRACT

Hybrid Composite Plate (HCP) is a novel retrofitting element which is tailored to be significantly free of the shortcomings identified in commonly used strengthening systems (e.g. FRP and TRM), mainly durability and connection reliability issues. HCP utilizes the synergetic advantages of Strain Hardening Cementitious Composite (SHCC) and Carbon Fiber Reinforced Polymer (CFRP) for the retrofitting of deficient RC members. Sufficient bearing capacity of SHCC offers the possibility of using mechanical fasteners to attach HCP to the RC substrate. This paper reviews the development of HCP and discusses its retrofitting performance based on experimental tests executed on the RC beams strengthened in shear or flexure and on the repaired RC beam-column joints. Moreover, following the results of the single-lap shear experimental tests and finite element analyses performed on the connection models, some aspects of HCP-RC connection are highlighted.

KEYWORDS
FRP, SHCC, repair and strengthening, flexural and shear, beam-column joint, bond stress-slip.

INTRODUCTION

A combination of Strain Hardening Cementitious Composite (SHCC) and FRP was recently adopted in the development of a thin prefabricated retrofitting element, designated Hybrid Composite Plate (HCP) (Esmaeeli, 2015). HCP was tailored in response to the shortcomings of the existing strengthening systems such as FRPs or textile reinforced mortars (TRMs). Bond degradation, premature debonding or detachment, vulnerability to impact loads and vandalism are examples of the drawbacks limiting a more extended use of FRP systems. Mechanical anchors, made of metallic or FRP materials, are proposed by several researchers to prevent/delay FRP debonding/detachment (Grelle and Sneed, 2013). Besides a high cost, the risk of corrosion and the rupture of FRP due to stress concentration at the anchored regions are disadvantages of using metallic fastening elements. Anchors made of FRP are susceptible to the drawbacks of the FRP-bonded systems. It is believed that Near Surface Mounted FRP (NSM-FRP) is less prone to the shortcomings identified in externally bonded (EB) FRP systems. Nevertheless, this solution has lower application versatility than EB-FRP (e.g. dependency on cover thickness and soundness, and on the width of the element). In the case of TRMs, generally premature debonding at the interface between the matrix and the retrofitted member or between the matrix and the fibers, restricts their retrofitting efficiency (Triantafillou and Papanicolaou, 2005). Furthermore, installing several layers of fabrics increases the in-situ workmanship job, which results in higher retrofitting costs. Moreover, excessive deflection and wide crack openings adversely affect the efficacy of the TRMs in enhancing serviceability performance of the concrete structures. Prefabricated plates made of fiber reinforced cementitious composites (FRCs), attached to the RC elements by means of adhesive, anchors or a combination thereof, have also been attempted as durable retrofitting alternatives (Alaee and Karihaloo, 2003, Misir and Kahraman, 2013). However, the retrofitting efficacy of these plates is restricted to an optimum strip thickness. This paper reviews the development of HCP and its retrofitting applications following the six years investigations performed in the scope of the PhD thesis of the Author since late 2009 (Esmaeeli, 2015).

MATERIAL-STRUCTURAL CONCEPT OF HCP

HCP combines the potential structural effectiveness of prefabricated SHCC reinforced with CFRP, in retrofitting of RC structures. As illustrated in Figure 1, CFRP is bonded onto the face of the SHCC plate, either in the form of sheets, designated as HCP(S), or in the form of laminates (bonded onto the pre-swan grooves), designated as HCP(L).
The thickness of SHCC plate may vary from 15 mm to 25 mm. SHCC is a composite made of a cementitious matrix reinforced with short randomly dispersed fibers, capable of developing a higher tensile strength than their first cracking strength. An appreciable amount of ductility under tensile loading is one of the most desired characteristics of SHCC, which originates from the formation of multiple diffused fine cracks before the SHCC attains its ultimate tensile strength. If SHCC is properly designed, before reaching its ultimate tensile strength, a self-controlled crack width mechanism retains the average crack width fine enough, generally below 100 µm. Thus, this cracked SHCC remains sufficiently impermeable. However, the ultimate tensile strength of SHCC is usually less than two times of its matrix tensile strength. CFRP, the other HCP constituent, has a high tensile strength with an almost linear-elastic response up to its tensile rupture. Hence, CFRP offers a negligible ductility. Therefore, as depicted schematically in Figure 2, HCP utilizes the synergetic advantages of CFRP and SHCC, namely strength and ductility, in the retrofitting of RC structures. Thanks to the bearing capacity of SHCC, this thin prefabricated plate can be attached to the concrete substrate by means of only mechanical anchors. However, depending on the retrofitting demand, the HCP-RC connection can be made of only adhesive or a combination of adhesive and mechanical anchors.

The FRP bonded face of HCP is placed in contact with the retrofitted RC member. This means that SHCC acts as a protective cover for the CFRP constituents. Therefore, this system is expected to endure a relatively higher temperature than a conventional FRP system. Furthermore, up to very close to the rupture strain of CFRP material only fine diffused cracks form in the SHCC, which potentially assures a long-time performance for the HCP’s constituents. Hence, as compared to the FRP strengthening techniques, HCP is more suitable for the repair applications or for the strengthening of RC members subjected to an aggressive environment, a relatively high temperature or considerable temperature variations, and the risk of vandalism or impact. As the detachment of HCP connected to RC elements by means of both adhesive and anchors is expected to be prevented or at least significantly delayed, this system offers an enhanced connection reliability, specially suitable for the strengthening of RC elements demanding a remarkable upgrade in their load carrying capacity. This technique is also appropriate where the concrete cover has a poor quality, since anchors are expected to transfer a proportion of the interlayer shear stresses to the element’s core, beyond the level of the main steel reinforcements. Furthermore, the proposed technique is independent of the concrete cover thickness. If increasing the shear capacity of the RC member is the strengthening objective, the notable contribution of SHCC plate in resisting shear stresses is combined with tensile contribution of FRP elements, which significantly enhances the shear strength of the RC member. In comparison to TRM, a superior bond strength at the interface of HCP constituents (CFRP and SHCC), and at the interface of HCP and concrete is attainable.
DEVELOPMENT OF HCP AND ASSESSMENT OF ITS RETROFITTING EFFICACY

Development of HCP was attempted in the scope of the PhD research work of the author, which was started in late 2009 and finalized in 2015.

**SHCC Characteristics**

The SHCC developed for the fabrication of HCP was composed of a finely graded cementitious matrix reinforced with randomly dispersed short Poly Vinyl Alcohol (PVA) fibers. 2% of PVA fibers (fraction of the total composite volume) were mixed into a fresh cementitious matrix which was rheologically adjusted to achieve a self-compacting composite. Portland cement type I 42.5R, silica sand and fly ash - materials available at the north Portugal – were used as the constituents of the matrix. These ingredients were proportioned taking into account the micromechanical demands of PVA-SHCC (Wang, 2005). Detailed information about the development of SHCC, including mix proportions, mixing procedure and methods of rheological and mechanical characterizations, can be found in (Esmaeeli et al., 2012) and in Chapter 4 of (Esmaeeli, 2015). Figure 3 depicts the typical tensile stress-strain response of the SHCC used in the development of HCP, which was characterized by means of direct tensile testing of thin SHCC prisms. As the typical mechanical properties of the PVA-SHCC, used in this study, were in average 2.75 MPa, 3.71 MPa, 18420 MPa and 35.2 MPa, respectively, and was about 1.54%.

![Figure 3 typical tensile stress-strain response of the SHCC used in development of HCP](image)

**Strengthening of Shear-Deficient RC Beams**

The results of the first series of three-point bending tests executed on shear strengthened RC beams showed that adhesively bonded HCP<span>1</span> plates are significantly efficient in increasing the shear strength and ductility of RC beams (Esmaeeli et al., 2013). However, the detachment of the HCP<span>1</span>, although much delayed as compared to that of the externally bonded CFRP sheets, highlighted the need for a further improvement in HCP-RC connection, such as adding mechanical anchors. Following this observation, author also hypothesized the concept of HCP<span>L</span>, SHCC plate with CFRP laminates bonded onto the cut grooves on its surface, to assure that: a) compatible materials, the strengthening plate (SHCC) and the concrete, are bonded to each other, b) a more efficient interfacial bond between CFRP and SHCC can be achieved (such as in NSM-FRP), and c) the post-cracking strain-hardening of SHCC contributes to stress transfer between cracked SHCC and CFRP-laminate. Figure 4 demonstrates the test setup, the configuration of the shear-deficient as-built beam and the strengthened ones along with their descriptions for the beams tested in the second series of the experiments. Moreover, two types of Plate-RC connection were studied: adhesively bonded versus connection by means of a combination of adhesive and chemical anchors. The force-deflection results obtained from these tests are shown in Figure 5. Moreover, the crack pattern at the failure of the beams strengthened with HCP<span>L</span> or SHCC plates, among others, are shown in Figure 6. Additional information about this experimental work, including material properties and the details of the strengthening systems, is available in section 4.5 of (Esmaeeli, 2015). Independent of the connection system, both types of HCPs were capable to alter the shear-tension failure mode, occurred in the other specimens, to a flexural failure mode. However, due to the scale-effect, utilizing the full shear strengthening potential of the HCPs was restricted to this flexural failure mode. Although the improvement in the deflection ductility in the case of HCP strengthened beams was more notable than that of the beam strengthened with adhesively bonded U-shaped CFRP sheet, the extent of this enhancement was a function of the type of the connection. The main advantage of adding chemical anchors to the adhesive-based connection of the strengthening plates, within the context of the study performed on the retrofitting of short-span beams, was hindering the sliding of the beams’ longitudinal tension bars. The effect of this restricted sliding was reduction in the rate of the post-peak load-decay in the case of retrofitted beams failed in flexure (HCP retrofitted ones), and higher shear capacity in the case of the retrofitted beam failed in shear (the...
beam retrofitted with SHCC plates). Visual inspection of the specimens at the end of their testing revealed that the debonding of HCP occurred only in the case of the adhesively bonded HCP\textsuperscript{LJ} plates.

Figure 4 the configuration of the shear-deficient as-built beam (left), the configuration of the strengthened beams (middle), and specimens' labels and descriptions (right)

Figure 5 load-deflection response of beams strengthened with adhesively bonded schemes along with the as-built beam (left), and beams strengthened with adhesively bonded plates versus those with a combination of adhesive and chemical anchors (right) - for the beams' nominations refer to Figure 4.

Figure 6 crack pattern of the beams strengthened with the SHCC or HCP\textsuperscript{LJ} plates at their failure considering the two different types of connection: adhesive versus a combination of adhesive and chemical anchors

**Flexural Strengthening of RC Beams**

The load-deflection responses obtained from four-point bending tests executed on a series of flexurally under-reinforced beams retrofitted by attaching HCP\textsuperscript{LJ} to their soffit and on the as-built beam are depicted in Figure 7. The description of each beam is presented at the right side of this figure. The number of CFRP laminates in the structure of HCP\textsuperscript{LJ} (two vs four laminates), the type of HCP\textsuperscript{LJ} connection (chemical anchor, adhesive, or a combination thereof), and the layout of chemical anchors (staggered vs one row) were the investigated parameters. As compared to the results of the as-built beam, all of the adopted strengthening schemes resulted in a superior response in terms of the load and deflection at the onset of cracking, yield load of the tension steel bars, and ultimate load. In the case of the connection by means of only chemical anchors, the bearing capacity of HCP\textsuperscript{LJ} was sufficient to mobilize up to 74% of the tensile strength of two CFRP laminates to the RC beam, corresponding to 72% gain in the ultimate load of the as-built beam (the splitting failure in HCP\textsuperscript{LJ} is shown in the left side of Figure 8). This type of connection resulted in deflection ductility of 5.3 which was the largest as compared to the other types of connections. The staggered configuration of the anchors was more efficient than one row of them in delaying the progress of detachment in concrete cover (detachment path is shown in the right side of Figure 8). This layout of anchors in combination with epoxy adhesive provided a significant increase of 167% in the flexural load carrying capacity as compared to that of the as-built beam (corresponding to 83% mobilization of tensile strength of four CFRP laminates), with a satisfactory deflection ductility of about 4.0. There were no cracks visible,
to an unaided eye, on the surface of the strengthening layer, while several cracks along the loading span on the lateral faces of the beams were already existed.

<table>
<thead>
<tr>
<th>Label</th>
<th>Strengthening Scheme</th>
</tr>
</thead>
<tbody>
<tr>
<td>FB_R</td>
<td>N/A (as-built reference beam)</td>
</tr>
<tr>
<td>FB2</td>
<td>HCP_L with 2 CFRP laminates</td>
</tr>
<tr>
<td>FB4</td>
<td>HCP_L with 4 CFRP laminates (one row of anchors)</td>
</tr>
<tr>
<td>_B</td>
<td>attached using only anchors</td>
</tr>
<tr>
<td>_G</td>
<td>adhesively bonded</td>
</tr>
<tr>
<td>_BG</td>
<td>attached using both adhesive and anchors</td>
</tr>
<tr>
<td>_Phi10</td>
<td>one row of 10 mm diameter anchors</td>
</tr>
<tr>
<td>_Phi8</td>
<td>staggered configuration of 8 mm diameter anchors</td>
</tr>
</tbody>
</table>

Figure 7 load-deflection responses from bending tests on flexurally strengthened beams and the as-built one

Figure 8 splitting failure in SHCC along the anchors in beam FB2_B (left), and detachment progress in concrete cover of beam FB4_BG_Phi8 (right)

Repair of Severely Damaged Beam-Column Joints

To experimentally assess efficacy of HCPs for enhancing seismic performance of RC structures, two full-scale severely damaged interior beam-column joints were repaired by attaching 25 mm thick HCPs to their critical regions, and they were tested. While cross-shape HCP_L plates were attached to the front and rear faces of both specimens, to one of the specimens also L-shaped HCP_O plates were fixed to the lateral faces of the beams and columns at each corner. These plates were attached by means of adhesive and chemical anchors. Similarly to the test configuration at their virgin state, these repaired specimens were subjected to simultaneous constant axial load and lateral reversal cyclic load, imposed on the top of their column. According to the tests results, both retrofitting schemes resulted in a superior performance in terms of hysteretic response, lateral load carrying capacity, energy dissipation capacity, beams flexural resistance and degradation of the secant stiffness as compared to the test results in their virgin state. A four-sided retrofitting solution altered the joint shear failure occurred in the other specimen and the virgin ones, to the failure caused by forming plastic hinges at the interfaces of the beams and the joint, and consequently the longitudinal CFRP laminates on the beams region ruptured (the failure mode of each of these repaired specimens are shown in Figure 9). By adopting a four-sided retrofitting solution, an increase of 48.3% in the lateral load carrying capacity, 84% in energy dissipation capacity and 22.2% in initial secant stiffness, as compared to the seismic performance of its virgin state, were obtained.

Figure 9 Damages at the strengthened beam-column joints at their failure, two-sided solution (left), and four-sided solution (right) (Esmaeeli et al., 2015)

HCP_L-RC Connection

A combination of experimental tests and finite element modellings was utilized to firstly characterize the local bond stress-slip laws at the interface of the SHCC and CFRP laminate and at the interface of the HCP_L and
concrete. The calibrated FE model was then extended to identify the pull load capacity of the HCP\textsuperscript{L(L)} as a function of the bond length and the stiffness of CFRP laminate, and further to optimize the parameters affecting the connection between HCP\textsuperscript{L(L)} and concrete - chapter 7 of (Esmaeeli, 2015). Following the results of this investigation, it was found that for an adhesively bonded HCP\textsuperscript{L(L)}, cohesive failure of SHCC is the prevailing failure mode. This mode of failure restricted the connection capacity to 65% (24kN) of tensile capacity of a single CFRP laminate. Thus, the use of anchors to further mobilize the tensile capacity of HCP\textsuperscript{L(L)} was found inevitable. If cohesive failure in SHCC is prevented, a CFRP-SHCC bond length of 90 mm is sufficient to mobilize the full tensile capacity of a single CFRP laminate (S&P laminate CFK 150/2000: 1.4×10 mm²). The effective CFRP-SHCC bond lengths for single- and double-CFRP configurations were identified as 200 and 250 mm, respectively, which correspond to maximum pull forces of 39.5 kN and 57.5 kN. Taking into account both the FE results and the practical considerations, a distance of 25 mm between groove and anchors is proposed. In the case of HCP\textsuperscript{L(L)} adhesively bonded to the RC block, with two rows of single-CFRP laminates, addition of a single anchor (200 mm far from the bottom edge of the plate) increases the pull force capacity by 41%. However, doubling the stiffness of the CFRP-laminates adversely affects this efficiency (only around 20% increase). Finally, for an HCP\textsuperscript{L(L)} composed of two rows of double-CFRP laminates bonded on an SHCC plate of 18 mm in thickness, both optimized width and height are 250 mm. This HCP\textsuperscript{L(L)} can mobilize 108 kN pull force to the concrete substrate if it is connected by means of adhesive and two chemical anchors.

CONCLUSIONS

The assessment of the retrofitting efficacy of HCP confirmed its suitability to upgrade variety of structural demands of the RC elements, such as increasing the shear and flexural capacity of RC beams, and enhancing seismic performance of beam-column joints. The bearing capacity of HCP is sufficient to be attached by means of chemical anchors and adhesive, ensuring a high HCP-RC connection reliability. Moreover, from the observations of the experimental tests it was found that the cracks widths in the SHCC, at least up to the serviceability limit of the flexurally strengthened beams, remain fine enough to be sufficiently impermeable. Thus, the constituents of cracked HCP are well protected against severe environmental conditions. The SHCC is also expected to protect the adhesive used to bond FRP and HCP from a relatively high temperature, temperature variations, and vandalism. However, the long-term durability, creep and fatigue performances of the HCP strengthened RC members need to be investigated.

ACKNOWLEDGMENTS

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ABSTRACT

The use of Glass Fibre Reinforced Polymer (GFRP) bars for internally reinforcing concrete structures has been growing to overcome common problems of steel reinforcement, such as corrosion. The Dundas Point Boardwalk in Applecross, Western Australia is subject to a harsh marine environment and remedial works to the foundation system which included corroded steel piles were focused on achieving an extended design life and addressing the ongoing corrosion issues encountered in marine environments. In stage one of the boardwalk repair, sixteen concrete columns and eight concrete footings were constructed, all internally reinforced with GFRP bars. The design of the footings and columns was performed using Canadian Code CAN CSA S806-12 and ISIS Canada’s guide ‘ISIS Educational Module 3: An Introduction to FRP-Reinforced Concrete.’ As the boardwalk decking was constructed using steel supporting members, attention was paid to detailing the concrete to steel connections in order to avoid moisture ingress and bimetallic corrosion. This connection was designed using GFRP threaded bolts embedded into the concrete columns and connecting the supporting steel members. Contractor feedback highlighted the ease of workability with the GFRP bars and all research suggests that there is no reason that a design life in excess of 75 years for the concrete elements cannot be achieved.

KEYWORDS

GFRP bars, compression members, durability, reinforcement, corrosion.

INTRODUCTION

Background of Dundas Point Boardwalk

Dundas Point Boardwalk is approximately 250 metres long, between 2.5 and 7 metres in width, and is located on the southern side of the Swan River Estuary near the north and western sides of Dundas Point in the City of Melville, Western Australia. The majority of the boardwalk is in the tidal zone of the Swan River and receives constant wetting and drying of its support structure. The boardwalk is used for foot and bike traffic only.

The boardwalk was designed and constructed in the 1990’s utilising steel universal column (UC) piles encased in concrete a minimum of 200mm below ground level at the time of construction. This supported steel framing on which a timber deck was laid. All steel members including piles and steel framing were hot-dipped galvanised.
Steel thickness measurements were performed on a selection of the steel piles. They were assessed in accordance with AS 1627.0-1997: ‘Metal finishing - Preparation and Pretreatment of Surfaces.’ All piles were classified as suffering from Grade H corrosion – ‘Large portion of surface is covered with rust, pits, rust nodules and non-adherent paint, pitting is visible.’ The results of the thickness measurements were extrapolated through a detailed non-destructive visual inspection to give an assessment of all 76 of the boardwalk piles. These results showed that 48% of the piles were between 20-30% corroded, 34% of the piles were 30-40% corroded and 5% of the piles exhibited more than 40% corrosion.

Due to funding limitations, it was decided to only remediate a 41 metre section of the boardwalk at the southwestern corner with the remainder of the required works programmed into the City of Melville’s future works programme.

Background to GFRP

Fibre Reinforced Polymers (FRP) are composite materials that can be used to strengthen concrete structures. They are made of fibres of a particular material selected, embedded in a polymeric resin. The most common fibres used in FRPs are glass, carbon or aramid. FRPs can come in woven sheets, which attach to the outside of reinforced concrete structures to offer strengthening, usually for remediation purposes. FRPs are less commonly produced as reinforcing bars.

Advantages of GFRP bars include; having high tensile strength, being corrosion resistant, nonmagnetic and lightweight with low thermal and electrical conductivity. This suite of characteristics is useful in many situations. GFRP bars are well suited for use in corrosive environments, in structures required to have a very long design life, in hospitals near MRI machines, for example, and provide easy workability because they are lightweight.

Disadvantages of GFRP bars include; no yielding before failure, low transverse strength compared to steel, low modulus of elasticity and possible durability issues of glass fibres in high alkaline environments. These characteristics need to be understood with guidelines to manage risk associated with these properties in concrete.

In 2001, the American Concrete Institute (ACI) released their first standard detailing recommendations of the use of FRP bars in reinforced concrete (RC). “Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars” is the most current standard and was published in February 2006. ACI Committee 440 has chosen not to offer recommendations on the use of FRP bars in compression members due to the lack of experimental data at that time. The Canadian Code CAN CSA S806-12 does include the use of GFRP in concrete compression members but ignores the contribution of GFRP when considering axial capacity.

Objectives

Experimental research at University of Western Australia was undertaken by the writer in 2012 who compared the performance of axially loaded concrete columns reinforced with GFRP bars to traditional steel reinforcement “The Study of FRP Strengthening of Concrete Structures to Increase the Serviceable Design Life in Corrosive Environments.” The results concluded that the same axial capacity could be achieved using the same area of reinforcement providing the GFRP transverse reinforcement spacing was lowered to approximately half that of the steel equivalent. That study led to the decision to construct concrete columns in an active project not only to achieve a corrosion resistant solution which would be economically beneficial, but to also monitor the performance of the columns, validating the findings of previous research.

INITIAL DESIGN PHASE

Constraints

The project brief stipulated that remedial works undertaken must yield a similar appearance as the existing boardwalk. This therefore set column sizes at 300mm in diameter and required them to have a black exterior. The same size and arrangement of steel framing was to be used to support the decking. Whilst keeping to these aesthetic requirements, the Client required a design life of over 25 years. The portion of boardwalk being replaced was 23 years old.

In the existing northeastern section of the boardwalk, there is pile support for the riverside of the boardwalk, with the steel cross beam which connects being supported in a recess into the limestone retaining wall on the land side. This connection was unstable as the retaining wall was rotating due to erosion issues below the boardwalk. This meant that new supports were needed to avoid reliance on the retaining wall for support.
Proposed Solution

The proposed solution included the replacement of pile supports whilst keeping a portion of the embedded steel UC pile to provide added anchorage of the element into the ground. The existing concrete encasement to piles was to be fully removed and replaced with a durable GFRP reinforced concrete solution. Whilst the existing steel UC pile was engaged, its requirement was designed out in the final design. This meant that a new footing would be required to support the concrete columns.

It was proposed that to eliminate the corrosion vulnerable column to superstructure connection, GFRP bolts would be cast into the new columns. This was deemed necessary as the steel framing for the decking was to be replaced and treated with an ultra-high build epoxy painting system so as to achieve a system that could be maintained over time and give a long design life of 20-25 years in a harsh marine environment.

Where the boardwalk cross beams were embedded in the limestone retaining wall adjacent, it was proposed to support them on new GFRP reinforced concrete columns identical to the replacement columns.

In the original construction there was a 300mm deep layer of sand extending up from the base of the concrete encasement of the steel piles. Over time this had been eroded away, in some cases exposing the limestone base below. This meant that two arrangements of footings were required to be designed, one founding on sand areas of the site and one founding on limestone.

Figure 2 Steel Cross Beam Embedded in Retaining Wall.

Figure 3 Proposed Column and Footing Design.
**FINAL DESIGN**

**Columns**

The final design of the columns included six 15.88mm GFRP longitudinal bars (1.68% area of reinforcement) with ligatures at 100mm centres made from 9.53mm GFRP bars. Although the bars are non-corrosive, it was decided that a pour blocker would be used in a 40MPa concrete mix to minimise concrete degradation. This was used with the intent of achieving a design life of the columns in excess of 75 years.

As there was the requirement for the new columns to look the same as the existing, permanent fibre cement formwork 300mm in diameter was specified and then painted black. This allowed for easy forming and yielded a neat appearance.

The existing steel piles were nominated to be cut back to leave an 800mm lap to be embedded in the replacement column. Following this they were to be cleaned to a Class 2.5 finish according to AS 1627.4 using grinding to remove the bulk rusted layers followed by wet abrasive blast cleaning or a blasting / washing / blasting sequence to achieve effective removal of soluble salts. Surface chlorides were tested to ensure that they were less than 50 micrograms per square centimetre. Where otherwise, they were to be re-washed with inhibitor solution then re-blasted prior to coating with a corrosion inhibiting priming paint.

**Footings**

The strip footings were designed to be able to support the loads through the new columns without the need for the engagement of the existing steel pile. One strip footing is used to support a pair of columns. The footing is 2.5 metres long, as required due to the placement of the columns, 600mm wide and minimum 250mm deep. The footing has bottom reinforcement, with six, 15.88mm GFRP bars equally spaced over the long span and four, 15.88mm GFRP bars over the short span. Six starter bars extend from the footings which would lap onto the main column reinforcement.

Due to the variable site conditions, the strip footing could be slightly altered for each case. Where sand is present, the footing and column are to be constructed with 500mm of sand above the top of the footing to allow for future erosion, or until bedrock is encountered. Where bedrock is exposed, the footing is to be recessed 250mm into the bedrock.

**Connection to Steel Framing**

The connection detail between the new columns and the steel cross beams which support the decking is critical. As this zone was prone to the deposition of salts, GFRP bolts were assessed as the best material for fixing the columns to the steel. Four 24mm diameter GFRP threaded rods were anchored 1 metre into the new concrete column. The new steel UC cross beams which were fabricated had additional plates welded to the bottom flanges in order to allow for these bolts to be accommodated. Due to the high grade paint specification used, EPDM washers were used to protect the steel from the nut and washer fixings.

As there was a gap between the top of column and the underside of the steel cross beam, two ‘U-bars’ were fabricated from 15.88mm GFRP and cast into the top of the column. This allowed for a non-shrink structural epoxy grout to be used to infill the gap and be engaged by the remainder of the concrete column. The grout could then be tapered to the steel cross beam to minimise pooling of any moisture.
CONSTRUCTION PHASE

Footings

The existing steel UC piles were successfully cut back and cleaned via abrasive blasting and thorough wire brushing. Chloride tests were performed on five of the sixteen steel sections with all tests yielding results under the allowable 50 micrometres per square centimetre. The steel was then painted with the appropriate corrosion inhibitor within four hours of the chloride tests being undertaken. This ensured that no excess chloride deposition took place between the testing and painting phases.

The footings were formed and GFRP reinforcement and starter bars laid as per Airey Taylor Consulting drawings and specifications. All footings encountered bedrock and therefore all footings were recessed into the bedrock. Rock boulders would be placed at the base of the footings to assist in limiting erosion, protecting the retaining wall structure adjacent to the boardwalk. The Contractors noted the ease of laying the reinforcement due to its lightweight. Plastic zip ties were used to tie bars together.

Columns

Once the footings were cast and cured, the column reinforcement cages, which were assembled on site, could be installed and the permanent formwork positioned and propped correctly. Timber plates were used to position the GFRP threaded rods at the correct height whilst the columns were poured. The installation of GFRP reinforcement was completed in the same way that traditional steel reinforcement is.

Finished Boardwalk

Following installation of the steel framing, a composite decking was installed and the original hardwood handrails were reinstalled. After the remediation of the boardwalk reclaimed beach sand was placed over the top of the footings with a geotextile placed on top of that with additional rock armouring and beach re-nourishment. This was to aid in protecting the surrounding structures.

The construction phase was carried out in line with the final design and construction administration was performed by the writer throughout to ensure this was the case. The Client was very pleased with the outcome and the fact that the remediated portion of the boardwalk looked very similar to the existing, although now with a design life of 75 years for the columns and footings, minimizing their future maintenance costs in a very inaccessible area.
The Contractor had no issues with the placement of GFRP and advised that it was similar to working with steel reinforcement but lighter.

![Figure 7 New (left) vs Old Boardwalk (right).](image)

**COST ANALYSIS**

Cost estimates were gathered from steel reinforcing suppliers in Perth for the fabrication and delivery of equal volumes of steel reinforcement compared to the GFRP bars specified. The steel estimates were approximately $4,000 whereas the fabrication and delivery (via air freight) of GFRP came in at $7,866. The price supplied for the same amount of GFRP but delivered via sea was $6,810.

The difference of approximately $4,000 is less than 1.5% of the total project cost which tendered for the amount of $275,772.28 + GST. This was considered to be negligible, whilst the advantages in corrosion resistance and extended time prior to first maintenance are significantly increased.

**CONCLUSIONS**

Previous experimental research performed by the writer in 2012 concluded that concrete compression members reinforced with GFRP bars gave comparable capacities to concrete columns reinforced with traditional steel. This research was the basis of the design of the Dundas Point Boardwalk.

The use of GFRP reinforcement in compression members is not recommended by current guidelines and the Dundas Point Boardwalk is the first project in Australia known to have used the technique. This is considered a big step in the acceptance of GFRP as reinforcement not only in compression members but in all concrete structures in Australia. Whilst there are many projects overseas which utilize this material, Australia has been slow to adopt it, and as all things new, people want to see it used in projects before confidence can grow.

The additional 1.5% of the total project cost was assessed as a negligible outlay in order to achieve concrete members which are expected to give good in service duty for over 75 years. The lack of maintenance required will allow for this slight start-up price increase to be recouped multiple times over the life of the structure.

The Client was receptive to the use of GFRP reinforcement at the concept design phase of the project, and the Contractor had no negative feedback whilst using the material and commended the workability of the product. This project, the ease with which it was executed, and the attenuated durability expected will give confidence to others to adopt the material.

**REFERENCES**


FLEXURAL DESIGN OF LIGHT RAIL GUIDEWAY GIRDER USING ULTRA-HIGH-PERFORMANCE FIBRE-REINFORCED CONCRETE AND GLASS-FIBRE-REINFORCED POLYMER

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ABSTRACT

Many of the challenges preventing widespread use of Glass-Fibre-Reinforced Polymer (GFRP) composites in the construction industry can be overcome by using GFRP in combination with Ultra-High-Performance Fibre-Reinforced Concrete (UHPFRC). To illustrate the potential of this material combination, a design study of a light-rail guideway girder is undertaken. It is shown that a girder designed with UHPFRC and GFRP can equal the performance of a conventional prestressed concrete girder while weighing half as much and potentially simplifying the construction process. UHPFRC and GFRP acting compositely are together able to overcome many of the challenges to designs using of each material on its own. GFRP structures require very high safety factors for sustained loads to protect against creep effects, and their low elastic modulus and lack of plasticity often mean that serviceability requirements dictate design rather than ultimate capacity. Compositely acting UHPFRC adds stiffness and keeps sustained stresses in the GFRP low at service load levels, while providing some plasticity at ultimate limit states. Fabrication of voided cross-sections entirely composed of UHPFRC is difficult given the low viscosity of wet UHPFRC, and the very slender structural elements enabled by its high-performance properties lead to questions regarding resilience. Compositely acting GFRP has the potential to significantly enhance the ductility and resilience of UHPFRC, and it can act as a structural, stay-in-place form to facilitate fabrication.

KEYWORDS

GFRP, UHPFRC, hybrid structures, guideway structures.

INTRODUCTION: GFRP IN CONSTRUCTION

The high strength and corrosion resistance of GFRP make it an attractive material for use in the construction industry. However, relatively low stiffness, nearly elastic behaviour to failure, and susceptibility to creep and fatigue present challenges for engineers designing with GFRP. Most current applications of GFRP in the construction industry use GFRP in place of steel in traditional reinforced concrete or steel girder-concrete deck composite systems. Such systems always face challenges stemming from the low stiffness of GFRP in comparison to steel, and its lack of any tensile yield plateau to give ductility to the structure.

This suggests that the structural systems that have evolved using steel and concrete may not be as well suited to hybrid composites involving GFRP (the term “hybrid” is used to refer to composites which consist of two or more compositely acting composite materials). The high stiffness and ductility of steel complement the brittle, relatively soft mechanical properties of concrete in a mutually beneficial way. The properties of GFRP do not seem to be so compatible with conventional concrete.

Combining materials into structural composites gives designers the ability to use the strengths of one material to offset the weaknesses of the other. Both GFRP and conventional concrete are approximately elastic to failure in tension, both are susceptible to creep, and both have a relatively low stiffness. As a result, any structural system using these two materials will inevitably suffer from these same weaknesses. In seeking a material partner for GFRP with which it can achieve a symbiosis similar to that which exists between steel and concrete, one must look for a material that can replace the stiffness and ductility that GFRP lacks. Ultra-High-Performance Fibre-Reinforced-Concrete (UHPFRC) possesses these enhanced properties and promises to be a good match for GFRP.

While many other researchers have explored structural components using polymer composites acting compositely with conventional concrete (Chang, 2006), only a few research projects to date have explored the potential of this material combination. Keller et al. (2007) tested UHPFRC-GFRP sandwich panels with a lightweight concrete...
core. El-Hacha and Chen (2012) have tested hybrid composite beams consisting of pultruded GFRP sections acting compositely with UHPFRC and Carbon-Fibre Reinforced Polymer (CFRP).

The potential of GFRP-UHPFRC hybrid composites will be demonstrated here through a brief description of the UHPFRC mix that has been developed at the University of Toronto, followed by a design study for a light-rail guideway girder.

ULTRA-HIGH-PERFORMANCE FIBRE-REINFORCED CONCRETE

UHPFRC is a dense cementitious composite reinforced with short steel fibres. The composition of the UHPFRC developed at the University of Toronto (Shao & Gauvreau, 2015) is detailed in Figure 1 along with its basic mechanical properties, which are used in the design study to follow. It has a lower compressive strength than many mixes, a compromise that has been consciously made in favour of lower cement content and a lower volume of fibres.

Despite its many impressive characteristics, UHPFRC has, like GFRP, seen relatively limited use in construction. Aside from cost there are a number of challenges preventing its widespread acceptance:
1) It is a highly self-consolidating mix. Its low viscosity is required in order to achieve a good distribution of fibres, but becomes problematic for the fabrication of voided cross-sections. The void form must be fixed in place in some way, and there will be hidden surfaces and enclosed volumes where air could become trapped, forming unwanted voids.
2) Related to these difficulties, and because the fibre distribution and tensile strength can be influenced by the method of casting, UHPFRC is best suited to precast construction. However, a simple and effective means of connecting precast UHPFRC components does not yet exist.
3) The resilience of UHPFRC structures can be called into question due to the very slender elements that its high cost demands, and its high performance permits. Girder webs and flanges, which could be 100 mm or less in thickness, would be susceptible to damage from accidental impacts on a construction site.

These problems can be overcome by combining UHPFRC with a second material, such as GFRP. The GFRP can act as a stay-in-place form to simplify fabrication, it can provide a means of transmitting forces across construction joints, and it can add reinforcement to enhance the structure’s resilience against impacts.

DESIGN STUDY: A LIGHT-RAIL TRANSIT GUIDEWAY GIRDER

The preliminary design of a guideway girder to support a light-rail transit line provides an effective example to illustrate the potential of GFRP-UHPFRC hybrid composite structures. The girder has a straight alignment, is simply supported, and spans 30 m. Only the flexural behaviour of the girder is studied in detail. A cross-section designed and developed by the authors using UHPFRC and GFRP is shown in Figure 2e. For comparison, four other designs are also considered and presented in Figure 2:
a) Prestressed concrete box girder
b) Steel U-girder with a compositely acting concrete deck
c) GFRP acting compositely with conventional concrete rather than UHPFRC
d) UHPFRC post-tensioned with external unbonded tendons

In all cases, a perfect shear connection between materials is assumed, producing fully composite behaviour. Achieving this behaviour in reality may not be a trivial matter. However, the assumption allows a plane-sections analysis that is sufficiently accurate for the purpose of comparing the global behaviour of the designs.

The light rail vehicle loading is simplified to a uniformly distributed load of 18.5 kN/m for fatigue and service checks, and 22.8 kN/m for ultimate capacity checks. A superimposed dead load of 5 kN/m is included to account for barriers, emergency walkway, rails, conduits, and all other permanent attachments. The self-weight of the girders ranges from 12.4 kN/m to 25.5 kN/m. For a 30 m simple span, applying load factors of 1.2 for dead loads and 1.7 for live loads, the factored moment demand ranges from 6710 kNm for the GFRP-UHPFRC girder to 8480 kNm for the prestressed concrete girder. The fatigue moment range is 2080 kNm.

The primary service requirement for a guideway girder is that it must have adequate stiffness to avoid problems with vibration. If this requirement is satisfied, it is likely that static deflection and crack control limits will also be satisfied. Detailed analysis of train-structure dynamic interaction can be very complex, but a simple check of natural frequency can be used at the preliminary design stage to ensure that resonance between the dominant structural and loading frequencies is avoided. A train car with a bogie spacing of 12 m, travelling at 25 m/s (a typical maximum speed for light rail systems) would create a cyclic load effect with a frequency of $25/12 = 2.1$ Hz. Other characteristic lengths, such as the train and span lengths, will be longer, contributing lower-frequency components to the fluctuating load. Therefore, if the natural frequency of the guideway structure is kept above 2.5 Hz, the risk of encountering problems with resonance later in the design process will be low.

Ultimate limit states requiring attention at the preliminary design stage are the static ultimate capacity, fatigue, and in the case of GFRP structures, creep failure. Buckling is neglected in this study; stiffening elements could be added as necessary to the slender steel webs of (b) and GFRP webs of (c) and (e).

The relative efficiencies and problems associated with each structural system can be compared with reference to the moment-curvature response for each cross-section, shown in Figure 3. These diagrams are derived without material resistance factors in order to allow a clear comparison between materials governed by different regulatory codes. Each of the designs is briefly discussed below.

(a) Prestressed Concrete

This cross-section is based on the design used for the Vancouver Expo Line guideway girders (Mills, 1986). It has a typical moment-curvature response for a prestressed concrete girder, maintaining the uncracked section stiffness under service loads before cracking and ultimately failing in a ductile manner by yielding of the prestressing strands. By properly proportioning prestressed and mild steel reinforcement it is generally possible to design a section, like this one, that efficiently meets both serviceability and ultimate limit state requirements. Since this cross-section has been successfully used for guideway structures, it serves as a baseline for evaluating other design concepts.

(b) Steel-Concrete

A steel U-girder acting compositely with a concrete deck is able to satisfy the strength and stiffness requirements with a 20% reduction in weight and a span-to-depth ratio of 26.1 compared to 21.9. However, the design is governed by the natural frequency limit, and careful detailing would be required to avoid fatigue concerns. The ultimate capacity of the section is well beyond the ULS demand – the increased lightness and slenderness has been purchased at the cost of a less efficient use of materials. Decreasing the capacity without reducing the stiffness is not possible due to practical limits on the thicknesses of the webs and flanges of a welded girder.
Figure 2 Cross-sections for a 30 m simply supported guideway girder

(a) Prestressed Concrete
\[ w_{aw} = 25.5 \text{ kN/m} \]
\[ I_t = 0.301 \text{ m}^4 \]
\[ E_dI_t = 6020 \text{ MNm}^2 \]
\[ M_0 = 9700 \text{ kNm}^2 \]
\[ f_1 = 2.7 \text{ Hz} \]
\[ L_H = 21.9 \]
\[ A_{\text{concrete}} = 1.04 \text{ m}^2 \]
\[ A_{\text{prestress}} = 4760 \text{ mm}^2 \]
Total girder weight = 78 000 kg

(b) Steel Girder with Composite Reinforced Concrete Deck
\[ w_{aw} = 20.0 \text{ kN/m} \]
\[ I_t = 0.0276 \text{ m}^4 \]
\[ E_dI_t = 5520 \text{ MNm}^2 \]
\[ M_0 = 20 100 \text{ kNm}^2 \]
\[ f_1 = 2.6 \text{ Hz} \]
\[ L_H = 26.1 \]
\[ A_{\text{concrete}} = 0.600 \text{ m}^2 \]
\[ A_{\text{steel}} = 0.0738 \text{ m}^2 \]
Total girder weight = 61 200 kg

(c) GFRP Girder with Composite Reinforced Concrete Deck
\[ w_{aw} = 16.9 \text{ kN/m} \]
\[ I_t = 0.1081 \text{ m}^4 \]
\[ E_dI_t = 4320 \text{ MNm}^2 \]
\[ M_0 = 36 100 \text{ kNm}^2 \]
\[ f_1 = 2.5 \text{ Hz} \]
\[ L_H = 16.7 \]
\[ A_{\text{concrete}} = 0.600 \text{ m}^2 \]
\[ A_{\text{GFRP}} = 0.1425 \text{ m}^2 \]
Total girder weight = 51 700 kg

(d) UHPFRC Prestressed with External Unbonded Tendons
\[ w_{aw} = 16.2 \text{ kN/m} \]
\[ I_t = 0.1029 \text{ m}^4 \]
\[ E_dI_t = 5145 \text{ MNm}^2 \]
\[ M_0 = 9550 \text{ kNm}^2 \]
\[ f_1 = 2.7 \text{ Hz} \]
\[ L_H = 30.0 \]
\[ A_{\text{UHPFRC}} = 0.622 \text{ m}^2 \]
\[ A_{\text{prestress}} = 5040 \text{ mm}^2 \]
Total girder weight = 49 700 kg

(e) GFRP with Composite UHPFRC
\[ w_{aw} = 12.4 \text{ kN/m} \]
\[ I_t = 0.236 \text{ m}^4 \]
\[ E_dI_t = 9440 \text{ MNm}^2 \]
\[ M_0 = 14 120 \text{ kNm}^2 \]
\[ f_1 = 3.9 \text{ Hz} \]
\[ L_H = 21.4 \]
\[ A_{\text{UHPFRC}} = 0.439 \text{ m}^2 \]
\[ A_{\text{GFRP}} = 0.0696 \text{ m}^2 \]
Total girder weight = 37 900 kg

Figure 2 Cross-sections for a 30 m simply supported guideway girder
Figure 3 Moment-curvature diagrams for guideway girder cross-sections. Left view shows ductile response; right is an expanded view of the service load range.

(c) GFRP-Concrete
This is as close to an all-GFRP solution as is possible. The concrete deck is required for rail attachment and would also help with geometry control of the track bed. The design is unsatisfactory in a number of ways. The section depth must be increased to 1.8 m in order to achieve the required natural frequency, which creates a span-to-depth ratio of 16.7, seriously compromising the aesthetics of the structure. This increased depth is necessary to control the effects of fatigue. As designed, the stress in the GFRP varies from 35 MPa to 65 MPa, or 13% of the ultimate tensile strength (UTS). The thickness of GFRP is 22 mm throughout; this could be increased to allow a higher span-to-depth ratio, but fabrication becomes more challenging with thicker GFRP plate.

(d) Prestressed UHPFRC
Setting aside the challenges discussed above of fabricating a UHPFRC box girder and ensuring its resilience, a UHPFRC girder, post-tensioned with external, unbonded tendons, could be an interesting option. Flexural behaviour very similar to that of the conventional prestressed concrete girder can be achieved with a girder that is 35% lighter. Furthermore, the UHPFRC girder has a span-to-depth ratio of 30, which would be a noticeable improvement in the slenderness of the structure. However, the costs of on-site erection and post-tensioning work would be similar to conventional construction. Therefore, the benefits may not outweigh the higher material costs, even if the afore-mentioned challenges could be overcome.

(e) GFRP-UHPFRC
This cross-section has been designed and developed by the authors at the University of Toronto. The GFRP shell would be supported at its end points in an unshored condition while the UHPFRC is cast. In this way, the GFRP carries all of the dead load, and the UHPFRC is able to remain uncracked under typical service loads. Because the UHPFRC provides most of the stiffness under service loads, it is possible to dimension the GFRP primarily for ultimate limit states, leading to a more efficient use of materials than is possible when designing with GFRP alone. Moment is plotted in Figure 3 relative to the curvature in the UHPFRC, which is zero under the dead load moment of 1970 kNm. The residual uncracked capacity is enough to withstand most of the live load stresses. Sections nearer to the supports will have a lower level of pre-load in the GFRP and may exhibit more cracking in the
UHPFRC under live load. A small amount of cracking in the UHPFRC would be beneficial to the damping of the system. The maximum permanent tensile stress in the GFRP from dead load is 85 MPa (17% of UTS), within recommended limits for avoiding creep rupture (Greenwood, 2002). Because the UHPFRC remains largely uncracked under live loads, the stress range in the GFRP is only 15 MPa (3% of UTS), and the maximum fatigue stress is thus 100 MPa (20% of UTS). These stress levels indicate that fatigue is likely to be a significant factor as the design is refined, but they are not so high as to preclude a viable solution for a reasonable service life. The extent to which fatigue is a problem also depends on the fatigue performance of UHPFRC, which is not yet well understood. A programme of fatigue testing of UHPFRC under cyclic tensile loads is underway at the University of Toronto to address this question.

The GFRP box shell could be fabricated by vacuum infusion, a process that has been successfully used for bridge girders in the past (Hurtado, 2012), and which allows girders to be fabricated with arbitrary curve or twist to accommodate the required track alignment. The casting of UHPFRC slabs into this shell would be a simple matter in comparison to casting a girder of the same geometry entirely out of UHPFRC.

CONCLUSIONS

GFRP-UHPFRC hybrid composite girders display structural behaviour that is superior to that of girders designed using either material on its own. The GFRP-UHPFRC system is able to match the performance of a prestressed concrete girder while cutting the girder weight in half. Specific benefits of the material combination include:

1) The stiffness and ductility of UHPFRC in tension is able to offset the relatively low stiffness and lack of ductility in GFRP.
2) By having UHPFRC carry most of the load under service conditions, fatigue stresses in GFRP are kept manageable without the need for far more material than is required for ultimate capacity.
3) Vacuum infusion of the GFRP allows fabrication of girders having arbitrary alignment or superelevation, an especially important consideration for guideway structures. The GFRP sets the geometry of the girder, and provides a stay-in-place form for the UHPFRC, overcoming the difficulties of casting complex geometries with a very low-viscosity concrete.

Work is on-going at the University of Toronto to continue to explore and characterize the behaviour of this promising new structural composite system.

ACKNOWLEDGEMENTS

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REFERENCES

EXPERIMENTAL AND NUMERICAL ANALYSIS OF AN FRP LIGHTHOUSE 
SUBJECTED TO FREE VIBRATIONS

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ABSTRACT

A lighthouse, 32 meters high and entirely made of fibre reinforced polymers (FRPs), was manufactured by ACCIONA Infrastructure in Madrid (Spain), and was installed in only two hours in the north extension of Valencia Port (middle east of Spain), in February 2015. This five-storey structure, which weighs 19 tons, is formed by eight carbon FRP circular hollow columns made by pultrusion and positioned at the vertices of an octagon. The 5 storeys are glass FRP and polyurethane octagonal sandwich panels made by resin infusion. An FRP spiral staircase is placed in the centre of the structure, going from its base to its top. To increase the lateral stiffness of the structure, between each couple of consecutive storeys, its carbon FRP columns are connected along the structure perimeter by horizontal glass FRP pipes which form in this way four octagonal rings. Once the FRP lighthouse was installed, its wind induced vibrations were recorded by a set of accelerometers strategically placed to determine its dynamic response. Previously, a numerical simulation of the structure was performed to assess its natural frequencies and mode shapes and to compare them with those determined experimentally.

KEYWORDS

FRP structures, free vibrations, dynamic response.

INTRODUCTION

In the marine environment, to minimize maintenance costs, the use of FRP materials for the construction of durable and lightweight civil structures is an attractive and promising alternative to traditional materials, such as steel or steel reinforced concrete. Because of the particular mechanical behaviour of FRP structures and the increased interest on this technology, many experimental and numerical research projects have been carried out in the last years, most of them focused on the static response of FRP structures, but very few in the field of dynamic response. In the case of the FRP lighthouse presented in this paper, the flexibilities of the connections between structural elements have an important influence on the flexural-torsional vibration response of the structure, so an adequate numerical model can be calibrated based on the experimental results from free vibration testing.

DESCRIPTION OF THE STRUCTURE

The lighthouse (Figure 1) is a five-storey structure supported by eight carbon FRP circular hollow columns whose centre lines, at the lower storey (S1), pass through the vertices of an octagon inscribed in a circumference of 4.15 m diameter and, at the upper storey (S5), through those of an octagon inscribed in a 3.75 m diameter circumference. These 32 m long columns, manufactured by pultrusion with epoxy resin (Figure 2a), have an outer diameter of 250 mm and a wall thickness of 20 mm. The 5 storeys, manufactured by resin infusion with vinylester resin (Figure 2b), are 200 mm thick sandwich panels with 10 mm thick glass FRP skins and a polyurethane core with a density of 70 kg/m³. The storeys are placed every 6 m and each one has a different octagonal geometry depending on its position in the structure. A spiral staircase is placed in the centre of the structure, going from its base to its top. The steps are manufactured by RTM and have a sandwich structure made of glass FRP skins and a polyurethane core (Figure 2c). Each step has a 200 mm rise and is formed by a ring with 500 mm inner diameter connected to a 900 mm length trapezoidal platform, with a variable tread width. The steps rings, vertically aligned along the lighthouse central axis, form a cylindrical space which is filled with reinforced concrete, providing a stiffening
core to the structure. To increase the lateral stiffness of the structure, the carbon FRP columns are connected by four octagonal rings placed between each two consecutive storeys. Each of these rings is formed by eight glass FRP pipes placed along the structure perimeter. The glass FRP pipes, manufactured by pultrusion, have an outer diameter of 190 mm and a wall thickness of 20 mm. The connections between columns and the horizontal glass FRP pipes are made by FRP rhomboidal diaphragms having a thickness of 42 mm (Figure 2d).

![Figure 1 The FRP lighthouse](image)

![Figure 2 Parts of the lighthouse during their fabrication: (a) columns; (b) storeys; (c) steps; (d) rhomboidal diaphragms](image)

The base of the lighthouse is a 4 m high reinforced concrete box with an octagonal prismatic shape. The lower ends of the lighthouse columns are embedded in the 1.10 thick box’s bottom slab. The 0.35 m reinforced concrete slab which forms the box ceiling is perforated to permit the eight carbon FRP columns and the central reinforced concrete core to pass through it. Neoprene bearing collars are placed between the FRP columns and this concrete slab to restrain the horizontal displacement of the columns at this level.
NUMERICAL SIMULATION

Model description

A three-dimensional finite element model of the whole structure is developed using SAP-2000 v16.1.1. The carbon FRP columns, the glass FRP pipes and the central core column are modelled with frame elements, while the storeys and the rhomboidal diaphragms are modelled with shell elements. Each column is fixed at its base, while its contact with the top slab of the concrete box is modelled with a set of linear elastic springs radially connected to the columns frame elements and having a stiffness equivalent to that of the neoprene bearing collar. Each structural element is characterized by the elastic parameters (\(E_x\), \(E_y\), \(G_{xy}\), and \(\nu_{xy}\)) and the specific weight taken from Table 1. Although carbon FRP columns and glass FRP pipes are orthotropic materials, the transverse elastic modulus \(E_y\) is not used in the model since these parts are modelled with frame elements. The spiral staircase steps are not modelled since it is assumed they do not contribute to the stiffness of the structure. Nevertheless their masses are considered adding them to the frame elements which model the concrete core below S5.

Table 1 Elastic parameters of the structural elements

<table>
<thead>
<tr>
<th>Element</th>
<th>(E_x) (N/mm(^2))</th>
<th>(E_y) (N/mm(^2))</th>
<th>(G_{xy}) (N/mm(^2))</th>
<th>(\nu_{xy})</th>
<th>(\rho) (kN/m(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon FRP columns</td>
<td>90740</td>
<td>-</td>
<td>35668</td>
<td>0.272</td>
<td>16.69</td>
</tr>
<tr>
<td>Glass FRP pipes</td>
<td>49860</td>
<td>-</td>
<td>19584</td>
<td>0.273</td>
<td>20.27</td>
</tr>
<tr>
<td>Storeys</td>
<td>2493</td>
<td>2493</td>
<td>308</td>
<td>0.084</td>
<td>2.50</td>
</tr>
<tr>
<td>Rhomboidal diaphragms</td>
<td>25840</td>
<td>22430</td>
<td>6003</td>
<td>0.383</td>
<td>18.33</td>
</tr>
<tr>
<td>Concrete core below S5</td>
<td>33620</td>
<td>-</td>
<td>14008</td>
<td>0.200</td>
<td>29.50</td>
</tr>
<tr>
<td>Concrete core above S5</td>
<td>28577</td>
<td>-</td>
<td>11907</td>
<td>0.200</td>
<td>25.00</td>
</tr>
</tbody>
</table>

The mechanical properties of each element are determined experimentally through static tests. In Figure 3, some of these tests are shown.

Figure 3 Static tests on structural elements (a) carbon FRP columns; and (b) glass FRP pipes

Modal analysis

A modal analysis of the numerical model of the structure is carried out to obtain the vibration modes and the corresponding frequencies, with the aim to compare them with those measured experimentally. The first ten vibration modes in the numerical model are presented in Table 2.

Table 2 First ten frequencies of the structure obtained numerically

<table>
<thead>
<tr>
<th>Frequency</th>
<th>(f_1)</th>
<th>(f_2)</th>
<th>(f_3)</th>
<th>(f_4)</th>
<th>(f_5)</th>
<th>(f_6)</th>
<th>(f_7)</th>
<th>(f_8)</th>
<th>(f_9)</th>
<th>(f_{10})</th>
</tr>
</thead>
</table>

EXPERIMENTAL ANALYSIS

In May 2015, three months after the installation of the lighthouse, wind induced vibrations in the structure are recorded by a set of accelerometers strategically placed to determine its dynamic response. Eight unidirectional DeltaTron Accelerometers Type 4508 are used, connected to its corresponding data logger Bruel&Kjaer LAN-XI 51.2 kHz - Type 3050. At each storey, but at the lowest one (S1), two unidirectional accelerometers are placed, one oriented along X-axis and the other oriented along Y-axis (Figure 4).
At storeys S3, S4 and S5 each pair of accelerometers is fixed to a 5 kg steel block placed on the storey top surface near one of the carbon FRP columns, as it is shown in Figure 4a. At storey S2 the accelerometers are fixed to small L profiles bonded to the storey top surface near the centre of the structure (Figure 4b).

Different data records are registered in order to compare the results of each measurement to detect possible random differences between them. The experimental vibration frequencies are determined by the Fast Fourier Transform (FFT), which converts the accelerations recorded in the time domain to the frequency domain. The measurements are made with a sampling frequency of 100 Hz. The choice of this frequency for data processing is set after analyzing several measurements with a higher initial frequency and after the observation that there are no excited frequencies in the structure higher than 50 Hz. As an example, Figure 5 shows the registered accelerations and the corresponding Fast Fourier Transform for the pair of measuring points (X-axis and Y-axis) in storey S2 during one of these data records. After processing all the data records, the first ten frequencies shown in Table 3 are almost the same in most measurements with very little differences between them, so it can be assumed that the first ten global modes of vibration of the structure correspond to these excited frequencies.
Table 3 First ten frequencies of the structure obtained experimentally

<table>
<thead>
<tr>
<th>Frequency</th>
<th>$f_1$</th>
<th>$f_2$</th>
<th>$f_3$</th>
<th>$f_4$</th>
<th>$f_5$</th>
<th>$f_6$</th>
<th>$f_7$</th>
<th>$f_8$</th>
<th>$f_9$</th>
<th>$f_{10}$</th>
</tr>
</thead>
</table>

From these results, the first five experimental modal shapes can be compared to those obtained numerically. The first vibration mode recorded experimentally, with a frequency of $f_{1,exp} = 1.309$ Hz, corresponds to the second vibration mode in the numerical analysis, a flexural mode with a frequency of $f_{2,num} = 1.241$ Hz. The second vibration mode recorded experimentally, with a frequency of $f_{2,exp} = 2.979$ Hz, corresponds to the third vibration mode in the numerical analysis, a torsional mode with a frequency of $f_{3,num} = 2.907$ Hz. The third, fourth and fifth vibration modes recorded experimentally, with a frequency of $f_{3,exp} = 3.922$ Hz, $f_{4,exp} = 6.178$ Hz and $f_{5,exp} = 9.307$ Hz, respectively, correspond to the fifth, sixth and ninth vibration modes in the numerical analysis, corresponding to a flexural mode with a frequency of $f_{5,num} = 3.679$ Hz, $f_{6,num} = 6.350$ Hz and $f_{9,num} = 9.375$ Hz, respectively. In Figure 6 the first five vibration modes in the numerical model with a frequency very similar to that obtained experimentally are presented, and are compare in Table 4.

Figure 6 First five vibration modes and natural frequencies in the numerical model of the structure corresponding to those obtained experimentally

Table 4 Comparison between frequencies of the structure obtained experimentally and numerically

<table>
<thead>
<tr>
<th>Frequency</th>
<th>Experimental</th>
<th>Numerical</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_1$</td>
<td>1.309 Hz</td>
<td>1.193 Hz</td>
</tr>
<tr>
<td>$f_2$</td>
<td>2.979 Hz</td>
<td>1.241 Hz</td>
</tr>
<tr>
<td>$f_3$</td>
<td>3.922 Hz</td>
<td>2.907 Hz</td>
</tr>
<tr>
<td>$f_4$</td>
<td>6.178 Hz</td>
<td>3.628 Hz</td>
</tr>
<tr>
<td>$f_5$</td>
<td>9.307 Hz</td>
<td>3.679 Hz</td>
</tr>
<tr>
<td>$f_6$</td>
<td>13.920 Hz</td>
<td>6.350 Hz</td>
</tr>
<tr>
<td>$f_7$</td>
<td>20.790 Hz</td>
<td>6.378 Hz</td>
</tr>
<tr>
<td>$f_8$</td>
<td>24.410 Hz</td>
<td>8.853 Hz</td>
</tr>
<tr>
<td>$f_9$</td>
<td>27.380 Hz</td>
<td>9.375 Hz</td>
</tr>
<tr>
<td>$f_{10}$</td>
<td>35.470 Hz</td>
<td>9.619 Hz</td>
</tr>
</tbody>
</table>
CONCLUSIONS

The experimental analysis carried out based on acceleration measurements has proven to be a good technique to obtain useful information about the structural behaviour of the lighthouse from the free vibration of the structure.

The analysis of the wind induced accelerations by FFT identifies the first five modes of vibration of the structure, and the corresponding experimental frequencies are 1.309 Hz, 2.979 Hz, 3.922 Hz, 6.178 Hz and 9.307 Hz. The frequency analysis is completed with a Finite Element Model, with frequencies very close to those obtained experimentally. This model allows identifying the experimental modal shapes, where the first mode corresponds to the first flexural mode and the second one to the first torsional mode, being the following modes the second, third and fourth flexural modes, respectively.

Although for a global structural analysis the use of isotropic linear elastic constitutive laws adopted for frame elements (carbon FRP columns, glass FRP pipes and central core column) is accurate enough, a more detailed analysis taking into account the shear deformability effect would be required, especially of those parts influenced by the stiffness of connections.

The use of FRP materials in the construction of singular structures in aggressive environments is a promising technique that has been validated through many projects worldwide, as for example the lighthouse presented in this paper. This structure has met the quality and structural requirements imposed to these type of projects, not only for its static behaviour, but also for its dynamic response.

ACKNOWLEDGMENTS

The authors wish to express their gratitude to the laboratory technicians and engineers of ACCIONA Infrastructure Research Centre of Madrid involved in this project, for their participation in the design, manufacture and testing of the structure.

It is worth mentioning that this structure has been awarded at the JEC World 2016 Innovation Awards in Paris in the Infrastructures category with the title "All-composite lighthouse marine navigation aid".

REFERENCES

DURABILITY ASSESSMENT OF FRP BARS EXTRACTED FROM EXISTING FRP BRIDGE STRUCTURES EXPOSED TO FIELD CONDITIONS

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ABSTRACT
Since the late 1990’s there have been a number of bridges in the United States that have been built or rehabilitated using FRP materials. A number of these projects occurred within the state of Missouri between 1999 and 2012. To assess durability behavior of existing bridges, some samples with reinforcing bar including GFRP bars are extracted from several older FRP bridges like Southview Bridge in varying climates to examine their performance after a decade or more under field conditions. This paper presents several analytical methods like Short Bar Shear (SBS) test, Glass Transition Temperature ($T_g$) test, and Resin Content Measurement [Burn off testing (i.e. resin)] for FRP rebars, and PH Measurement for concrete to evaluate the both physical and chemical properties of GFRP bars and concrete with time under field environmental conditions. This paper is intended to provide a possible path forward to collect important field data on FRP reinforced structures with a view to create a database of field data to better understand the long-term behavior of FRP bars used in the field.

KEYWORDS
FRP, SBS, $T_g$, burn off testing, PH measurement.

INTRODUCTION
Corrosion of steel reinforcement is a major problem with traditional steel reinforcement used in bridge decks. In the last decade, the non-corrodible glass fiber-reinforced polymer (GFRP) bars have made them more attractive to civil engineers and construction companies to replace conventional steel bars due to their lower cost, higher tensile capacity, and lower weight. They can mitigate or prevent the corrosion problem of reinforced concrete (RC) structures, especially members and elements such as bridge deck slabs exposed to aggressive environments (Ghatefar et al. 2014). GFRP composite materials cannot be corroded by nature like their steel counterpart. However, concrete structures reinforced with GFRP materials are still susceptible to other forms of deterioration due to harsh environments involving deicing chemicals, sulfate salts and alkalis, which can readily infiltrate concrete through cracks (ISIS Canada 2006).

The state of Missouri has had a number of FRP projects for both strengthening of existing deficient bridges and new FRP bridge construction due to their corrosion resistance, high strength to weight ratio and rapid installation processes. Validation of the long-term durability performance and the comprehensive development criteria/guidelines are needed before FRP system will gain widespread acceptance throughout the engineering and civil infrastructure community in the United States. Therefore, many of the Missouri demonstration projects like the Walker Street Bridge and the Southview Bridge have included on-going monitoring and load testing to validate these systems and demonstrate their long-term durability performance (Holdener et al. 2008).

Some concrete cores involving GFRP bars were extracted from the Southview Bridge slab deck using GFRP bars for top and bottom mats and CFRP reinforcement for internal post-tensioning of the bridge deck. This bridge is located in Rolla, Missouri (Southview Drive on Carter Creek), U.S.A. One lane of the bridge was already built using a conventional four-cell steel reinforced concrete box culvert. One lane and sidewalk needed to be added. This additional lane was constructed using FRP bars as internal reinforcement in 2004 (Fico et al. 2006).
There are limited results whether the GFRP bars are deteriorated after long-term field exposure to concrete. As a result, these results cannot explain completely the properties of GFRP bars extracted from concrete structures. In this paper, therefore, SBS test, Resin Content Measurement (Burn off Testing), Glass Transition Temperature ($T_g$) test for GFRP bars, and PH Measurement of concrete cylinders with different locations were performed to determine whether epoxy and concrete were deteriorative with time and to suggest a possible path forward to creating a database of suggested test methods on field collected samples. The secondary objective of this study is to further illustrate whether the properties of GFRP bars will change after several years exposure to concrete environment, and provide a direction or guideline of long-term performance of FRP system so that this system can be applied widely to civil engineering. It should also be noted since these structures were constructed, manufacturers have made improvements their FRP product so any observations noted in this work certainly may not be reflective of bar products produced today with improvements seen in the resin systems.

EXPERIMENTAL PROGRAMS

Short Bar Shear (SBS) Test

Number 6 GFRP bars (0.75 in. (19 mm) diameter) that were manufactured by Hughes Brothers were extracted from some cylinders that were from Southview Bridge in Rolla, Missouri. Figure 1 shows the samples of GFRP bars. According to limitations and requirements of ASTM D4475-02 (Reapproved 2008) (ASTM D4475-02 2008), Short Bar Shear (SBS) test was performed. The test set-up is illustrated in Figure 2. The purpose of this test is to measure the inter-laminar shear properties of GFRP bars, and to compare to the values that manufacturer reported. Shear behavior would change if resin or epoxy in GFRP bars is destroyed or deteriorated under seasonal in-situ field exposure.

![Figure 1 Samples of GFRP bars](image1)

![Figure 2 Set up of SBS test](image2)

Resin Content Measurement (Burn off Testing)

GFRP bar samples were cut and weighed approximately 0.011 lb (5 g). Burn off testing was conducted based on ASTM D2584-11 (ASTM D2584 2011). Firstly, some samples were weighed, then placed on a substrate and re-weighed. The samples on the substrates were heated in the muffle furnace at 600°C (1112°F) until all resin had disappeared. The samples were then cooled and weighed again including the substrate. Resin contents of these samples were calculated and compared to the original values. The purpose of this test is to determine whether epoxy of GFRP bars is deteriorative after long-term field exposure to concrete.

$T_g$ Measurement of GFRP bar

The glass transition temperature ($T_g$), an important physical property of the matrix, is not only an indicator of the thermal stability of the material but also is an important indicator of the structure of the polymer and its mechanical properties (Mufti et al 2007). The ideal temperature for heat treatment application depends on the thermal behavior of each composite, such as $T_g$ analysis and initial degradation temperature (Miyazaki et al 2009). The $T_g$, therefore, can successfully be used as a reference to sign the ideal heat treatment for photo-irradiated resin compositions. According to ASTM E1640-13 (ASTM E1640 2013), TA instrument was used to perform $T_g$ measurement. Some powders of FRP samples were obtained from interior and surface of GFRP bar to evaluate:

1. the thermal behavior of epoxy;
2. the difference between interior and exterior of GFRP sample.

The samples were heated until 200°C (392°F).
PH Measurement of Concrete

The initial pH value of concrete surface is approximately 13. The value at the exposed surface will fall due to the reaction of carbon dioxide from the atmosphere and alkalis in the concrete. The process is known as carbonation. Depth of carbonated concrete will continue to develop over time. Because the carbonated concrete can allow corrosion of reinforcing bars, it may be important to determine the depth of carbonated concrete. This test was followed ASTM F710 (Section 5.2.1) (ASTM F710 2011). First, the concrete surface was ground by sand paper, and cleaning the surface. Distilled water was placed on this surface. PH paper was used to measure pH value. It was compare with the pH value that concrete surface was not ground.

EXPERIMENTAL RESULTS AND DISCUSSIONS

Short Bar Shear (SBS) Test

In order to perform this test, the specimen length was 3.5 in. (89 mm) based on ASTM D4475-02. The Instron 4400 Series Universal Testing Machine (UTM) was used to do this test. The same rate of loading, 0.05 in./min (12.7 mm/min) was used. The specimen was center-loaded. The center-to-center distance between two supports was 2.25 in. (57.2 mm). The ultimate failure load and displacement of the inter-laminar shear test were recorded. Figure 5 illustrates the experimental load-displacement curve of GFRP sample.

![Figure 5 Load-displacement relationship](image)

The ultimate load at failure and final deflection were 2913 lbs and 0.274 in (6.96 mm) respectively. Figure 6 shows the failure specimen after the test. There was some deformation of the specimen directly under the loading head at the beginning of loading due to the low stiffness and strength of resin compared to fibers of this GFRP bar. This can explain that the curve of load-displacement have a lower slope at the beginning of this relationship, but there was an increase in slope after the original deformation as showed in Figure 5. Finally, the load decreased suddenly because the crack started from the mid-plane under the loading head. The failure of the specimen occurred. The GFRP bar could still carry some load because of flexural strength of the sample after ultimate failure. At the same time, the crack width also increased gradually. This specimen showed a vertical plane of failure (plane of load) with crack perpendicular to the cross-section of the rod as illustrated in Figures 6 and 7.

![Figure 4 Failure GFRP specimen](image)

![Figure 5 Cross section of GFRP](image)
According to ASTM D4475-02, shear strength was determined by the equation (1):

\[ S = 0.849 \frac{P}{d^2} \]  

(1)

where S is the apparent shear strength, P is the breaking load, and d refers to the nominal diameter of the FRP bar. Due to lack of the original test data of the SBS prior to construction of this bridge, this paper referred to the interlaminar shear strengths of control and in-service specimens that were extracted from Sierrita de la Cruz Creek Bridge (built in 2000, located 25 miles Northwest of Amarillo, Texas) to serve as a comparison. Table 1 illustrates the results.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>In-service specimen (2016)</td>
<td>Manufacturer (Control, 2000)</td>
</tr>
<tr>
<td>0.75</td>
<td>P (lb)</td>
<td>S (psi)</td>
</tr>
<tr>
<td></td>
<td>2913</td>
<td>4396.7</td>
</tr>
</tbody>
</table>

Conversion Units: 1 in = 25.4 mm, 1 lb = 4.448N, 1 psi = 0.0069 MPa

These GFRP bars were produced by the same manufacturer at different periods. Even if there were no original data of SBS to refer for Southview Bridge, the in-service samples still suggests that shear strength of GFRP bar shows a decreasing trend compared to average results of control for the Sierrita de la Cruz Creek Bridge. It may suggest that the GFRP bar was deteriorative under the long-term exposure to concrete environment. However, the chemical composition of fiber/resin, different ratio of fiber and resin, or other parameters may vary for different production lots. The shear strength of control samples for Sierrita de la Cruz Creek Bridge can only be considered as a reference. Additional SBS tests will be needed to investigate the interlaminar shear property of GFRP bars so it is suggested that a suite of tests be done at the time of construction to set a baseline for future sampling.

Burn off Testing

First of all, the muffle furnace without GFRP samples was heated until 600°C (1112°F). Secondly, three small samples with substrates that were obtained from two GFRP bars were put into the muffle furnace to heat until all resin had disappeared. The duration of this test was about 7 hours. The content of resin was calculated. Samples prior and after burn off testing are illustrated in Figure 6.

![Figure 6 Samples prior (left) and after (right) burn off testing](image)

Detailed information of GFRP samples before and after test is illustrated in Table 2.

<table>
<thead>
<tr>
<th>Sample ID</th>
<th>Weight (g)</th>
<th>Weight with substrate before test (g)</th>
<th>Weight with substrate after test (g)</th>
<th>Weight of resin (g)</th>
<th>Percentage of resin (%)</th>
<th>Percentage of fibers (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample 1</td>
<td>5.57</td>
<td>130.29</td>
<td>128.76</td>
<td>1.53</td>
<td>27.5</td>
<td>72.5</td>
</tr>
<tr>
<td>Sample 3</td>
<td>5.4</td>
<td>199.07</td>
<td>197.59</td>
<td>1.48</td>
<td>27.4</td>
<td>72.6</td>
</tr>
<tr>
<td>Sample 4</td>
<td>4.66</td>
<td>272.56</td>
<td>271.28</td>
<td>1.28</td>
<td>27.5</td>
<td>72.5</td>
</tr>
</tbody>
</table>

Conversion Units: 1 g = 0.0022 lb
From Table 2, it can be observed that the percentages of resins have no difference between these samples. This paper also referred to the test data of the control specimens (80.5%) and in-service samples (79.7%) of Sierrita de la Cruz Creek Bridge. The average fiber content of GFRP samples for the Southview Bridge shows a lower fiber content compared to the control and in-service samples of Sierrita de la Cruz Creek Bridge. However, they were still above the minimum fiber content standard requirement of 70% by mass based on AC 454 (AC 454 2015).

**T<sub>g</sub> Measurement of GFRP bar**

Moisture in the matrix would reduce T<sub>g</sub> of the resin through plastification if the Van der Waals bond between the polymer chains is broken. The swelling stresses are able to result in permanent damage in the epoxy of GFRP bar such as matrix cracking, hydrolysis, and fiber-matrix de-bonding when the composite material uptakes moisture or alkalis present (Mufti et al 2007).

Differential Scanning Calorimetry (DSC) was used to evaluate the T<sub>g</sub> of the resins of surface and interior of the GFRP bar. It was found that the surface and interior of GFRP bar presented temperatures of 95.3°C (203.5°F) and 92.3°C (198.1°F) respectively.

**PH Measurement of Concrete**

Two different locations of concrete were selected to measure the pH values. The concrete of the first spot that was not ground by sand paper was the transition zone between concrete and GFRP bar. The fresh distilled water was put onto this surface. The pH value of this location was evaluated by pH strip and was about 9 as illustrated Figure 7.

In addition, the second location was selected as illustrated Figure 8. This concrete surface was ground and cleaned. The same fresh distilled water was still utilized and put onto the ground surface. The pH value of this location was 12 showing variation in the measured values.

The pH value of the ground surface was in agreement with the report from University of Miami (Gooranorimi et al 2016).
CONCLUSIONS

Durability of GFRP bars in existing FRP bridge structures exposed to the field environment is studied by using physical and chemical tests including SBS test, burn off test, and $T_g$ measurement of GFRP bars, and pH measurement of concrete. Concluding remarks are summarized as follows:

- According to ASTM D4475, short bar shear (SBS) test is performed in order to study resin properties of GFRP bars exposed to concrete environment. The horizontal shear strength results may provide a measure of resin damage. Micelli and Nanni (2004) studied transverse properties of GFRP rods. The results showed that failure loads of these rods decreased after they were exposed to alkaline solution. In this study, therefore, GFRP bars might be affected by hydroxyl ions (OH$^-$) in concrete. It is necessary that more SBS tests of GFRP bars need to be conducted to further investigate this property of resin to obtain more accurate results. Furthermore, FTIR evaluation will be conducted as supplemental investigation.

- Fiber content by mass should be more than 70% based on the report of manufacturer (Hughes Brothers), which was verified through this evaluation post installation.

- According to the manufacturer report, the glass transition temperature ($T_g$) or the temperature at which the resin changes from a “glassy state” and begins to soften is 110°C (230°F), which is higher than the results measured in this study. It is necessary that more investigation such as EDX microscopic studies be undertaken to better understand any changes in the chemical characteristics.

These measurements above cannot prove whether in-service GFRP bars have deteriorated after 11 years of service as concrete reinforcement in the Southview Bridge. In order to understand sufficiently their properties, SEM and EDX analyses will be performed to investigate whether there are some damage of glass fiber, resin, and fiber-matrix interface and identify existing chemical elements in GFRP bars as phase II work. In addition, Fourier transform infrared (FTIR) spectroscopy will be conducted in subsequent study to investigate the amount of OH$^-$ in GFRP sample.

REFERENCES

BRIDGE RETROFIT WITH FRP MATERIALS – FIFTEEN YEARS OF EXPERIENCE

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ABSTRACT

This paper summarizes the deployment of fiber reinforced polymer (FRP) composites in the form of fabric, laminates, and rod panels over the past fifteen years on twenty eight reinforced concrete, prestressed concrete, and steel bridges throughout Kentucky. The field applications of FRP composites include: (1) retrofit of PC beams in shear using unidirectional carbon fabric; (2) flexural strengthening of concrete and steel beams using CFRP laminates; (3) retrofit of cracked ends of PC I-beams using carbon fabric; (4) retrofit of cracked PC box beams using carbon fabric; (5) retrofit of impacted PC beams with CFRP Rod Panels; (6) retrofit of cracked concrete pedestals using carbon fabric; and (7) retrofit of impacted concrete piers and pier caps using carbon fabric. This paper highlights eight case studies, each of which includes an in-depth discussion of the retrofit techniques used.

KEYWORDS

CFRP, beam strengthening, bridge retrofit, carbon fabric, laminates, rod panels.

INTRODUCTION

Fiber reinforced polymer (FRP) composites — especially carbon FRP fabric and laminates — are becoming the primary choice to strengthen damaged structural components or upgrade structures. FRP materials have several benefits, including a high strength-to-weight ratio, resistance to corrosion, can be adhesively bonded, and higher moduli of elasticity than concrete. The low transportation and handling costs of FRP materials, as well as their low application and labor costs, reduce overall project expenditures when they are used. Compared to traditional repair methods, FRP is less disruptive to regular service during the repair process. Furthermore, repairs that employ FRP contribute minimal additional dead load to the structure.

Table 1 lists, in chronological order, all of the bridge retrofit projects carried out by the authors. Specifics on the damage type, the repaired/strengthened member type, and retrofit material are provided as well. Several of the bridge retrofit projects highlighted in Table 1 include the first field application of CFRP Rod Panels (CRPs), the second field application of Ultra High Modulus (UHM) CFRP laminates for steel girder strengthening, and the first civil engineering applications of quasi-isotropic braided triaxial carbon fabric. The following sections discuss selected projects from Table 1 in more detail, where the applications have been categorized based on the type of deficiency and/or strengthening carried out along with the type of material used.

<table>
<thead>
<tr>
<th>Retrofit Year</th>
<th>Damage Type</th>
<th>Member Type</th>
<th>Bridge Identification</th>
<th>Retrofit Material</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>2001</td>
<td>Shear Cracks</td>
<td>PC Box Beams</td>
<td>KY 3297 over Little Sandy River, Carter Co. KY</td>
<td>Unidirectional Carbon Fabric</td>
<td>Simpson et al. (2006)</td>
</tr>
<tr>
<td>2003</td>
<td>Flexural Cracks, Cracked Pedestals</td>
<td>RC Beams, Concrete Pedestals</td>
<td>KY 3 over Big Sandy River, Lawrence Co. KY</td>
<td>CFRP Laminates</td>
<td>Choo and Harik (2007)</td>
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<tr>
<td>2007</td>
<td>Vertical Cracks at Beam Ends</td>
<td>AASHTO Type II PC I-Beams</td>
<td>1-65 overpass between Jacob St. and Gray St., Louisville KY</td>
<td>Unidirectional Carbon Fabric</td>
<td>Choo et al. (2013a)</td>
</tr>
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<td>2007</td>
<td>Cracked Pedestals</td>
<td>RC Pedestals</td>
<td>1-65 over Jefferson St. and Preston St., Louisville KY</td>
<td>Unidirectional Carbon Fabric</td>
<td>Choo et al. (2013a)</td>
</tr>
<tr>
<td>Retrofit Year</td>
<td>Damage Type</td>
<td>Member Type</td>
<td>Bridge Identification</td>
<td>Retrofit Material</td>
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<td>2007</td>
<td>Cracked Pedestals</td>
<td>RC Pedestals</td>
<td>I-65 over Main St. and Hancock St., Louisville KY</td>
<td>Unidirectional Carbon Fabric</td>
<td>Choo et al. (2013a)</td>
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<td>2007</td>
<td>Flexural Cracks</td>
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<td>Choo et al. (2013b)</td>
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<td>AASHTO Type II PC I-Beams</td>
<td>I-65 overpass between Chestnut St. and Gray St., Louisville KY</td>
<td>Unidirectional Carbon Fabric</td>
<td>Choo et al. (2013a)</td>
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<td>2010</td>
<td>Pier Column Deterioration</td>
<td>RC Column and Pier Cap</td>
<td>I-65 overpass over Muhammad Ali Blvd., Louisville KY</td>
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<td>Choo et al. (2013a)</td>
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<td>2010</td>
<td>Load Posting</td>
<td>Steel Beams</td>
<td>KY 32 over Lytles Creek, Scott Co. KY</td>
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<td>Peiris and Harik (2015a)</td>
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<td>2011</td>
<td>Over Height Truck Impact</td>
<td>RC Edge Beam</td>
<td>Caldwell Road over Blue Grass Parkway, Anderson Co. KY</td>
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<td>Peiris and Harik (2015)</td>
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<td>2012</td>
<td>Cracks at Beam Ends</td>
<td>PC Box Beams</td>
<td>KY 100 over CSX Railroad and South Railroad Street, Simpson Co. KY</td>
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<td>Monitoring Stage</td>
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<td>RC Pedestals</td>
<td>KY 81 Bridge, McLean Co. KY</td>
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<td>2013</td>
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<td>KY 55 over Majors Run Creek, Carroll Co. KY</td>
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<td>Monitoring Stage</td>
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<td>2013</td>
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<td>RC Beams</td>
<td>KY 80 over I-69, Graves Co. KY</td>
<td>CFRP Rod Panels and Triaxial Carbon Fabric</td>
<td>Monitoring Stage</td>
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<td>2013</td>
<td>Over Height Truck Impact</td>
<td>RC Beams</td>
<td>KY 11 over Cat Creek, Powell Co. KY</td>
<td>CFRP Rod Panels and Triaxial Carbon Fabric</td>
<td>Monitoring Stage</td>
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<td>2013</td>
<td>Concrete Deteriorization and Rebar Corrosion</td>
<td>RC Edge Beam</td>
<td>KY 11 over CSX Railroad and Strodes Run, Mason Co. KY</td>
<td>CFRP Rod Panels and Triaxial Carbon Fabric</td>
<td>Monitoring Stage</td>
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<td>AASHTO Type IV PC I-Beams</td>
<td>KY 11 over CSX Railroad and Strodes Run, Mason Co. KY</td>
<td>Unidirectional and Triaxial Carbon Fabric</td>
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<td>2014</td>
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<td>Pier Cap/Walkways</td>
<td>I-64 over River Road, Louisville KY</td>
<td>CFRP Rod Panels and Triaxial Carbon Fabric</td>
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<tr>
<td>2014</td>
<td>Concrete Deteriorization and Rebar Corrosion</td>
<td>Pier Column</td>
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<td>Monitoring Stage</td>
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<td>2014</td>
<td>Concrete Deteriorization</td>
<td>RC Beams</td>
<td>KY 583 over Blue Grass Parkway</td>
<td>Triaxial Carbon Fabric</td>
<td>Monitoring Stage</td>
</tr>
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</table>
SHEAR STRENGTHENING
KY 3297 Bridge, Carter County

A routine inspection on this 3-span bridge revealed that significant diagonal shear cracks up to 29 mm wide and 1.8 to 2.4 m long had formed in all precast prestressed box beams (Fig. 1) at both ends of Span 2. Subsequent inspections revealed that the shear cracks in Span 2 were propagating at an alarming rate, and new shear cracks were also developing in Spans 1 and 3. In addition, the re-evaluation confirmed that the box beams were under-reinforced in shear. The retrofit was performed in two phases: (1) crack repair; and (2) application of CFRP fabric. The application of CFRP fabric was expected to strengthen and compensate for the shear deficiency.

When the cracks were initially discovered, the bridge had an estimated remaining life expectancy of three to five years. The application of the CFRP system only required the use of light construction kits and tools; no heavy machinery was used throughout the entire process. Another positive aspect of this particular project was that the impact on daily traffic was kept to a minimum while work was being performed underneath the bridge. When the project was carried out in 2001, the cost for the repair and 3 years monitoring was US$ 105,000, whereas replacing the superstructure would have cost, at minimum, US $600,000. Fifteen years after the retrofit, no crack movement or new cracks have been reported on the bridge.

FLEXURAL STRENGTHENING
KY 3 (Louisa-Fort Gay) bridge, Lawrence County

The Louisa-Fort Gay Bridge is a 12-span continuous bridge structure consisting of composite concrete deck-steel girder spans and reinforced concrete (RC) girder middle spans that comprise an intersection for traffic coming from three different directions. Bridge inspection indicated that flexural cracks had developed in the RC girders due to heavy coal truck loads. Weigh-in-motion scales measured trucks that weighed in excess of 1000 kN. Further evaluation confirmed that most of the reinforced concrete girders were stressed beyond the allowable service limits specified by the American Association of State and Highway Transportation Official Specification (AASHTO, 1998). The amount of CFRP laminates needed to strengthen the beams was determined via moment-curvature analyses (Choo and Harik, 2007). Figure 3 captures the underside of Span 4 following the laminate application. Similar to the Carter County bridge, there were minimal impacts on daily traffic during construction. The cost savings were considerable — the retrofit and three years of monitoring cost just US $195,000. Alternatively, it would have cost over US $2 million to construct a new bridge. The experience gained through the project on
application of CFRP laminates over long spans, considering man power, equipment and work time of epoxy adhesives, was instrumental in the development of CFRP Rod Panels that facilitates modular construction.

KY 32 Bridge, Scott County

The KY 32 Bridge over Lytles Creek, is a single span steel girder bridge with a non-composite reinforced concrete bridge deck. A load posting of 124.6 kN (14 ton) was on the bridge before strengthening. While ultra high modulus (UHM) CFRP laminates were considered for improving the load carrying capacity of the beams, a preliminary analysis indicated that, due to the non-composite deck, the increase would be minimal and the laminates would not be used in an effective manner. To utilize the laminates more effectively, shear studs were post installed between the steel girders and the concrete deck to make the bridge deck act partially composite with the girders. The number and location of the shear studs were evaluated to obtain maximum partial composite action while using the fewest number of studs. After the shear studs were set into place, UHM CFRP laminates were applied to the top and bottom of the bottom flange of the steel girders (Fig. 3). Field tests were conducted before and after the installation of the shear studs to verify the effectiveness of post installed shear studs (Peiris and Harik, 2015a). Field tests were carried out after the laminate application to determine the increase in the load carrying capacity and the reduction in maximum deflection after the strengthening. The combination of post-installed shear studs and UHM CFRP laminates increased the bridge’s load rating factor from 0.46 to 1.21, which permitted the removal of the load posting.

PC I-BEAM END STRENGTHENING

I-65 Elevated Highway Bridges, Louisville KY

A number of vertical cracks located near or at the supporting piers developed on many of the AASTHO Type II prestressed concrete (PC) girders between Jacob Street and Chestnut Street. Preliminary investigations indicated that the cracks may have developed due to shrinkage and temperature loading. The cracks were especially apparent at piers with fixed restraints, where the axial shortening of girders is prevented. Prior to retrofitting, the two widest cracks on two PC girders were instrumented with linear variable displacement transducers (LVDTs) to monitor the vertical and longitudinal movements. The bridge was monitored over a period of six months. The readings indicated that the cracks were very active but stable. This permitted the design of a simple and cost effective retrofit that will be continuously monitored from a remote location. A single layer of unidirectional carbon fabric was applied along the length of the beam at the crack locations, while additional strengthening in the form of vertical U-wraps were added at locations with multiple cracks and concrete spalling. After the repair, the two crack locations that had been monitored previously were again instrumented with LVDTs to gauge the repair’s effectiveness (Fig. 4).

The repair caused no traffic disruptions and utilized no heavy equipment. The carbon fabric retrofit extended the service life of the structure. Alternatives to the retrofit included the construction of new piers, the extension of existing pier caps, or replacement of the damaged section of the I-65 expressway. Given the number of crack locations, any one of these alternatives would have cost more than US $1 million. The combined use of remote monitoring and using lightweight CFRP fabric for the retrofit was the most cost effective and efficient solution to repair the cracked girders.
PC BOX BEAM STRENGTHENING
KY 100 Bridge, Simpson County

The two span PC box girder bridge on KY 100 over the CSX railroad had multiple beams with several cracks close to the abutments and the center pier. The cracks were thought to be due to uneven settlement of the abutments/pier, leading to slight out-of-plane twisting of the superstructure. Most of the cracks were primarily diagonal, originating from the top of the beam. However, several vertical and horizontal cracks, as well as inclined cracks, were also present. While bi-axial carbon fabric was initially considered for the retrofit, in order to address the cracks in multiple directions, a quasi-isotropic braided triaxial (0°, +60°, -60°) carbon fabric was used to strengthen the beam. Figure 5 shows a retrofitted PC box girder following application of the carbon fabric. The triaxial carbon fabric facilitated the use of a single layer of fabric to achieve the required amount of strengthening in multiple directions, reducing construction time, material, and labor costs.

IMPACTED BEAM RETROFIT
Sunny-side Gotts Road Bridge, Warren County

An AASHTO Type III precast girder of the four span bridge on Sunnyside-Gotts road over Interstate 65 was damaged by an over-height truck impact. The damaged section spanned the two right lanes of the northbound three-lane interstate highway. The over-height truck impact produced considerable concrete spalling, severed two prestressing strands, and damaged several wires in two more strands (Fig. 6). The retrofit replaced the lost concrete and installed CFRP Rod Panels (CRPs) to replace the capacity loss due to the damaged prestressing strands. CRPs were attached to the structural member as externally bonded reinforcement. Each panel is approximately 1.2 m long and is made of a number of small diameter rods that are attached to a glass fiber mesh, which is intended to facilitate handling of the panels and to retain a uniform spacing between rods. In the field, the panels were connected through a finger joint lap-splice to form a continuous externally bonded reinforcement and fulfil the strengthening length. CRP 195 panels, with CFRP rods of 4 mm diameter able to carry 867 kN (195,000 lbs) per 300 mm (1 ft) width of panel, were selected for the strengthening.

Because the bridge was over a major interstate, the retrofit was carried out at night to minimize traffic disruption. The use of CRPs provided several advantages over regular CFRP laminates due to the location and nature of the retrofit being over multi-lane traffic. Because CRP construction is modular, the retrofit could be stopped after installation of any CRP panel as long as the splice length of the last panel was kept void of epoxy. This is advantageous when impact damage spans multiple lanes of traffic, where the retrofit can proceed in stages, with strengthening occurring over one lane at a time. Using the CRPs also meant application could be executed with a two-person crew working out of a truck mounted mobile platform or man-lift. This eliminated the need for a larger crew and multiple pieces of access equipment or scaffolding that would be required for a similar retrofit using CFRP laminates.

CONCRETE PEDESTAL STRENGTHENING
Pier 6, I-65 Elevated Highway Bridges, Louisville KY

Bridge Pier 6 is located at the intersection of Main Street and Hancock Street. The six interior pedestals of the eight reinforced concrete pedestals showed signs of damage; with one in particular experiencing severe loss of concrete cover and exposure of reinforcing steel. The damage to the concrete pedestals was attributed to de-icing...
agents falling through the expansion joints over the piers. The pedestals posed a unique challenge as they were irregularly shaped (i.e., neither circular nor rectangular). Their unique shape rendered traditional wrapping techniques obsolete because the expansion of the pads would not have been confined by such wrapping. To surmount this obstacle, a prism (i.e., one of the corners) was sheared off of the bearing pad so that it could be confined by the wrapping. The objective of using carbon fabric wrapping was to improve concrete confinement and thus increase the pedestal’s load carrying capacity. Figure 7 shows a pedestal after loose concrete had been removed.

**IMPACTED CONCRETE PIERS**

**Elrod Road Bridge, Warren County**

The pier column of Elrod Road Bridge over Natcher Parkway was impacted by a truck, which resulted in cracking of the column and pier cap. The primary retrofit involved wrapping unidirectional carbon fabric around the column. An additional layer of triaxial carbon fabric was wrapped over the joints of the unidirectional fabric for confinement (Figure 8). The strengthening is expected to increase the load carrying capacity of the column more than 300%, from 2448 kN when unconfined to 8105 kN for the confined column.

![Figure 7 Deteriorated pedestal](image1)

![Figure 8 Carbon fabric application](image2)

**CONCLUSIONS**

FRP composites were used to retrofit several bridges in Kentucky because they offered significant economic advantages over other retrofitting alternatives. Carbon fabric wrapping was found to be an effective method for retrofitting for shear, concrete piers, pier caps and pedestals, and deteriorated or damaged PC and RC beams. CFRP laminates proved to be an efficient strengthening technique for RC and steel beams where the underside of the bridge is easily accessible. CRPs were efficient for retrofitting impacted and deteriorated concrete bridge beams with limited access. Based on the success of the projects listed in Table 1, FRP retrofit, where applicable, has become the primary choice for retrofits in Kentucky.

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DEVELOPMENT OF GFRP TRUSSED INSPECTION PATH WITH 10-M LONG SPAN

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ABSTRACT

An inspection path for highway bridges has been developed using pultruded GFRP members. The proposed bridge inspection path is a trussed girder type with a sandwiched panel floor made of GFRP and urethane foam. In this study, the inspection path with 10-m long span was developed and fabricated in order to apply to a steel plate girder bridge with a long interval of cross beam. The validity and usability of structural design was investigated by various loading tests using a full-scaled model. As a result of static loading test, it was found that the deflection limitation of 1/600 for the span length was roughly satisfied, and the result of some vibration test demonstrated that the uncomfortable vibrations did not occur. Therefore, it was confirmed the developed trussed-girder-type inspection paths had the required rigidity and safety, and was feasible to the practical use.

KEYWORDS

GFRP, inspection path, trussed girder, sandwich panel slab, static, dynamic and impact loading test.

INTRODUCTION

In Japan, many highway steel bridges are suffering from aging and need maintenance procedure. Current maintenance system of bridges requires close visual inspection once every five years (MLIT 2014) and installation of inspection paths is required. In particular, an inspection path using pultruded GFRP trussed girder has been developed for the highway bridges in order to effectively utilize the advantages of FRP (Ishii et al. 2016). The proposed bridge inspection path is a trussed girder type with a sandwiched panel floor made of mainly GFRP and urethane foam to increase the rigidity and reduce the weight. In addition, extension of the distance between the panel points with the same length and angle of diagonal member makes it possible to be applicable to various span.

Recently, the application of a steel plate girder bridge with the long interval of cross beams as the distance between them of 10m is gradually increasing to reduce cost associated with fabrication and construction. Considering such circumstance, an inspection path that its span is expandable to maximum span of 10m is required.

Therefore this paper deals with the serviceability and the safety of the inspection path with 10-m long span in order to apply to a steel plate girder bridge with a long interval of cross beam.

OUTLINE OF TRUSSED INSPECTION PATH WITH 10-M LONG SPAN

Design conditions of inspection path

Table 1 shows the design conditions of the inspection path. Nippon Expressway Company determines the standards for a FRP inspection path by references to existing FRP inspection path and R & D (Furuya et al. 2015). In this study, this standards were applied and the design live load (3.5kN/m²), the vertical and horizontal loads to a handrail and the required size for serviceability are shown in Table 1. The value of the deflection limitation for
inspection paths is 1/100 for a span length in this standards. However, considering the serviceability, the value of 1/600 for a span length was used in this study. The value is also used for a general footbridge (JSCE 2011).

<table>
<thead>
<tr>
<th>Table 1 Design conditions of inspection path</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design live load</td>
</tr>
<tr>
<td>Vertical load for handrail (short-term load)</td>
</tr>
<tr>
<td>Horizontal load for handrail (short-term load)</td>
</tr>
<tr>
<td>Effective width</td>
</tr>
<tr>
<td>Handrail height</td>
</tr>
<tr>
<td>Distance between vertical struts</td>
</tr>
<tr>
<td>Deflection limitation under design live load</td>
</tr>
<tr>
<td>Applicable span length L</td>
</tr>
</tbody>
</table>

### Proposed trussed girder type with a sandwiched panel floor

Figure 1 shows the general view of GFRP trussed inspection path with 10-m span length. GFRP channel profiles of C75 (H75×B40×t5.0 mm), which are composed of roving glass fibres, are used for upper and lower chords, diagonal and vertical members. GFRP square pipes of SP60 (H60×B32×t4.0 mm) are used for a horizontal member which is non-structural member. All the GFRP members are pultruded.

The floor slab is made of lightweight sandwich panels and Figure 2 shows the cross sectional view of sandwich panel floor slab. It is fabricated as follows: First, two GFRP channel profiles of C75 as a lower chord place on both sides and 2 mm thick continuous-molding GFRP skin plates are fixed on the top and bottom of lower chords. Then, interior of the floor slab is filled with hard urethane foam as a core material. Finally, toe-plates cut out from GFRP channels of C100 (H100×B50×t5.0mm) are fixed to both edges of the upper surface of the floor slab to prevent instruments and tools from falling down. It also affects to reinforce the joints and increase section rigidity.

GFRP angle members cut out from GFRP channels of C75 are installed at the panel points of the lower chord to reinforce the joints. The panel points are connected with stainless steel (JIS SUS305) blind rivets and adhesives. It is effective method for connection of closed section. Table 2 shows the material properties by coupon tests. The material test was conducted on the basis of Japanese Industrial Standards (JIS).

### Table 2 Material properties by coupon tests

<table>
<thead>
<tr>
<th>Member</th>
<th>Tensile property</th>
<th>Compression property</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Strength σtu (N/mm²)</td>
<td>Elastic modulus Et (kN/mm²)</td>
</tr>
<tr>
<td>C75</td>
<td>600.0</td>
<td>39.5</td>
</tr>
<tr>
<td>C100</td>
<td>597.9</td>
<td>39.2</td>
</tr>
<tr>
<td>SP60</td>
<td>521.9</td>
<td>32.5</td>
</tr>
<tr>
<td>Skin plate</td>
<td>78.0</td>
<td>8.3</td>
</tr>
<tr>
<td>Hard urethane foam</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
PERFORMANCE VERIFICATION TEST FOR INSPECTION PATH

Evaluation of deflection under design live load

In order to evaluate the deflection under design live load, the static loading test was conducted as Figure 3. Sandbags equivalent to 3.5kN/m² were placed on the floor slab and its vertical deflection was measured.

Figure 4 shows the distribution of vertical deflection under design live load. The maximum deflection was 17.11mm at the center of the span. It was found that the experimental values were slightly larger than the analytical values based on FEA. However, the value of deflection limitation (L/600) for the span length was roughly satisfied.

Analytical evaluation of safety under design live load

In order to examine the safety of GFRP trussed inspection path with 10-m span length under design live load, FEA was conducted. According to the results of GFRP trussed inspection path with 6-m span length in previous study (Ishii et al. 2016), it has been found that relationships between the applied load and displacement at the center of the span are shown in Figure 5. When the applied load was approximately 50 kN, the shear failure occurred at the panel point of the upper chord as shown in Figure 6. Figure 5 shows that the analytical result is different from the experimental result after shear failure. Therefore, maximum load of GFRP trussed inspection path with 10-m span length was evaluated by the maximum shear strain of 8484 (×10⁶) in previous study. In addition, it has been found that the experimental axial strains at the panel point of the upper chord was sufficiently smaller than the failure strain in material test.

The deformation of GFRP trussed inspection path with 10-m span length under design live load is shown in Figure 7. The focused point in Figure 7 also shows the panel point of the upper chord where the maximum shear strain was confirmed. Figure 8 shows the analytical relationships between the applied load and the shear strain at the focused point. From the result in Figure 8, the applied load which was compatible with the shear strain of 8484 (×10⁶) was 60.47kN. Therefore, it was found that the predicted failure load was 60.47kN. Also, the predicted failure load was 2.94 times higher than the design live load of 3.5kN/m².
Vibration characteristics of inspection path

In order to examine the vibration characteristics of the inspection path, some vibration tests were conducted. Figure 9 shows set-up of the vibration test. Sandbags weighing 20kg, 40kg and 60kg respectively, were hanged at the upper chord level of 1.1m in height, and then dropped on the center of the floor slab as free vibration method. Natural frequency was measured with accelerometers at under surface of the floor slab and the upper chord.

The first natural frequency calculated by vibration tests is shown in Table 3. The minimum natural frequency was 5.43Hz, which was higher than that of uncomfortable vibrations to humans (the range of 1.5Hz to 2.3Hz). Therefore, it was confirmed that natural frequency of the proposed inspection path satisfied the vibration serviceability.

Safety of handrail member under static load

Using the inspection path for safety checks, workers will lean on or step on its handrail. Considering such circumstance, in order to examine the safety of handrail members, the vertical and horizontal static loading tests were conducted. Sandbags equivalent to 0.59 kN/m were placed between the panel points of the upper chord as Figure 10 (a). The maximum strain of each member under the vertical static load was measured. Moreover, the maximum strain of each member under the horizontal static load was also measured with the concentrated load equivalent to 0.39 kN/m. Its load was statically applied to the panel point of the upper chord by using a screw jack as Figure 10 (b).

<table>
<thead>
<tr>
<th>Mass of a sandbag</th>
<th>Floor slab (Vertical)</th>
<th>Upper chord member (Horizontal)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20kg</td>
<td>11.90</td>
<td>5.68</td>
</tr>
<tr>
<td>40kg</td>
<td>11.29</td>
<td>5.55</td>
</tr>
<tr>
<td>60kg</td>
<td>10.93</td>
<td>5.43</td>
</tr>
</tbody>
</table>
Table 4 shows the results of the static loading tests. It was confirmed that the measured strains were smaller than the failure strain of 15,000 (×10⁶) confirmed by material test. Therefore, the strength of the handrail member possessed sufficiently higher strength under the vertical and horizontal static load.

<table>
<thead>
<tr>
<th>Loading condition</th>
<th>Measured strain (×10⁶)</th>
<th>Location of Maximum strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical load</td>
<td>132</td>
<td>Handrail member</td>
</tr>
<tr>
<td>Horizontal load</td>
<td>-660</td>
<td>Panel point at lower chord member</td>
</tr>
</tbody>
</table>

**Resistance of handrail member to impact load**

In order to examine the resistance of the handrail member to impact load as the maximum applied load considering the accidental fall of inspectors, impact loading test was conducted as Figure 11. The conceptual diagram of impact loading test is shown in Figure 12. A sandbag weighing 85 kg was connected to the handrail through a safety belt and dropped from upper chord level.

As a result, Figure 13 shows the damage to a handrail member after impact loading test. A crack occurred at the corner located between the upper flange and the web of the upper chord. However, there were no failure of members and no damage to panel points. Additionally, the handrail member prevented the sandbag as an inspector’s weight from falling to the ground.

Table 5 shows the measured maximum strain under impact load. From the result in Table 4, each measured strain was smaller than the failure strain of 15,000 (×10⁶) confirmed by material test. Therefore, it was confirmed that handrail members possessed sufficiently safety for the fall of inspectors.
Table 5 Maximum strain at impact load

<table>
<thead>
<tr>
<th>Item</th>
<th>Measured strain ($\times 10^6$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intended handrail member</td>
<td></td>
</tr>
<tr>
<td>Upper chord</td>
<td>-3550</td>
</tr>
<tr>
<td>Horizontal bar</td>
<td>-1305</td>
</tr>
<tr>
<td>Panel point of lower chord member</td>
<td>-2122</td>
</tr>
<tr>
<td></td>
<td>-2096</td>
</tr>
</tbody>
</table>

CONCLUSIONS

In this study, the inspection path with 10-m long span was developed and fabricated in order to apply to a steel plate girder bridge with a long interval of cross beam. The performance verification test using a full-scale model was conducted to confirm the serviceability and the safety of the inspection path. The following conclusions were obtained:

(1) In static loading test under design live load, it was found that the deflection limitation of 1/600 for the span length was roughly satisfied. Also, the estimated strength of the proposed inspection path by FEA was sufficiently higher than the material strength under the design live load.

(2) In some vibration tests, the minimum natural frequency was higher than that of uncomfortable vibrations to humans. It was confirmed that the natural frequency of the proposed inspection path satisfied the vibration serviceability.

(3) In vertical and horizontal static loading tests, the strength of the handrail member possessed sufficiently higher strength under its design load.

(4) In impact loading test considering the accidental fall of inspectors, partial damages occurred at handrail members. However, it was confirmed that handrail members did not fall and possessed sufficiently safety for the fall of inspectors.

Therefore, it was confirmed the developed trussed-girder-type inspection paths with 10-m long span possessed the required rigidity and safety, and was feasible to the practical use.

REFERENCES


ABSTRACT

This paper presents a project in Hong Kong for bridge strengthening using fibre-reinforced polymers (FRP) and this is the first case approved by the local authority. Old structures often need repair and strengthening to stay in a serviceable condition and this paper reports on the strengthening of a 30-year old pre-stressed concrete bridge to satisfy the new building regulations in Hong Kong. Due to site constraints, a combination of post-tensioning and fibre-reinforced polymeric technology was used. In this paper, the design basis is presented and details for implementation are highlighted, including selection for materials, method of installation, fire proofing, full load testing and monitoring. To demonstrate the success of the works, full load-testing and several long-term monitoring techniques were designed and installed including optical fibre strain measurement. Since the completion of the strengthening works, long term monitoring has been carried out. The long term monitoring results show that the FRP is in a good condition after 5 years.

KEYWORDS

FRP, bridge, strengthening, long-term monitoring.

INTRODUCTION

Swire Bridge is located at the Main Campus of the University of Hong Kong. This bridge was constructed in 1970s. It consists of a two-lane carriageway and a footpath and serves as the main vehicular and man access to the eastern part of the campus from the central part. The total length of the bridge is 52.5m. It consists of three continuous spans supported by two end abutments and two intermediate piers. The main span is 22.5m and the two side spans are 15m. The bridge deck is constructed with prestressed concrete with internal tendons passing through the bridge deck. Bridge piers and abutments are conventional reinforced concrete structures and they are supported by bored piles.

Swire Bridge is an emergency vehicular access and truck access. Current trucks and emergency vehicles can weigh 28ton or more. However, the bridge is designed to resist a live load of 20 ton originally. Therefore there is a need of upgrading the bridge to accommodate the latest live load requirement.

A design check was conducted to assess the existing capacity of the bridge, including overall stability and load capacities of different structural elements. Design check results showed that while the bridge was of sufficient overall stability and loading capacity to resist the increased load in the ultimate limit state, the bridge deck failed to pass the serviceability limit state requirements. For example, the tensile stress would excess of the allowable value of 2.4MPa. It was anticipated that the bridge deck would be of excessive deflection and have potential cracking problem. Excessive deflection and cracks would affect the durability of the structure by allowing corrosive substances, e.g., oxygen and moisture, penetrating into cover of concrete. The risk of corrosion of steel reinforcement would increase. Severe steel corrosion would lead to delamination of concrete cover and spalling problems, reducing the life of the bridge deck. In extreme case, if the existing prestressing tendons are corroded, the prestressing force would reduce, causing detrimental effects to the loading carrying capacity.

To control the potential deflection and cracking problems in the bridge deck at service load, a combination of prestressing and fibre-reinforced polymeric (FRP) technology was used, the latter of which formed the first project case approved by the Hong Kong building authority. Due to the pioneer nature of the project, full load-testing and several long-term monitoring techniques were designed and installed including optical fibre strain measurement. In this paper, the design basis is presented and details for implementation are highlighted, including selection for
materials, method of installation, fire proofing, full load testing and monitoring. Finally the test results are presented and long-term monitoring data discussed, from which some preliminary conclusions are drawn.

**STRENGTHENING DESIGN CONSIDERATIONS**

**Strengthening Methods Selection**

During the project design stage, various strengthening methods were considered with a view to increase the bridge’s performance in deflection and cracking at service load. The potential strengthening methods include span shortening, installation of additional steel frame or additional concrete layer, strengthening with fibre-reinforced polymeric (FRP), additional prestressing, etc. Each method has its own characteristics and hence strengths and shortcomings. During the selection process, preference was given to the method with minimal visual impact and ease of application. Span shortening and additional steel frame would cause substantial change in bridge’s appearance and hence are rejected. Additional concrete layer would affect road profiles of adjacent access roads and it is not preferred to have extensive modification works to these adjacent roads. FRP strengthening is the most preferred method since it can maintain the original appearance of the bridge and the application method is simple. However, FRP strengthening is able to resist additional load only. To reduce deflection and control cracks caused by existing dead loads, the method of prestressing is preferred. Therefore, after careful considerations, both FRP and prestressing technologies are selected as a combined strengthening solution. By adopting this strengthening method, the cracking and deflection of the bridge will be better controlled. Under extreme loading conditions, the bridge will not be subject to excessive deflection and cracking. Corrosive substances such as carbon dioxide and moisture cannot penetrate through concrete cover into steel reinforcement and prestressing tendons. The durability of the bridge is hence better assured and the service life is extended.

**Strengthening Materials**

The two common FRP materials for structural strengthening applications are glass fibre reinforced polymer (GFRP) and carbon fibre reinforced polymer (CFRP). Typical tensile properties of FRP materials can be found in design guidelines in Concrete Society – Technical Report No. 55 Design Guidelines for Strengthening Concrete Structures using Fibre Composite Materials, Second Edition. In general, tensile strength and modulus of CFRP are higher than those of GFRP. To satisfy the performance requirement, the higher stiffness material, i.e., CFRP, is preferred, so that the composite section has a higher overall stiffness against bending and hence deflection and crack control.

Consideration is also given to durability performance. Both GFRP and CFRP do not have corrosion problem since they are non-metallic materials. The identified deterioration mechanisms of FRP materials are degradation due to ultra-violet light and damage in case of fire. In this case, a fire insulation layer is used to protect the installed FRP against fire and block the ultra-violet light. Since that the durability performances of two FRP materials are similar but CFRP has a better loading performance then GFRP. Therefore CFRP is adopted for this project.

For the prestressing tendon, prestressing bars with higher tensile load capacity are selected. To prevent corrosion, heat shrinkable plastic tubes are adopted. The plastic tubes serve as physical barriers which block water and moisture from going into the steel surface so that corrosion will not occur.

**Strengthening Design**

According to Buildings Regulations of Hong Kong, any structural alternation and additional works shall be approved by Buildings Department. To obtain this approval, submissions with full justifications are needed. The required submissions include structural strengthening design, quality control proposal, materials’ durability data, etc.

This project involves a bridge structure and therefore the bridge’s structural analysis is in accordance with “BS 5400 – Steel, Concrete and Composite Bridges” and “Highways Department – Structures Design Manual”, both of which are design manuals common adopted in Hong Kong. For the design of FRP, since there is no available British Standards or Hong Kong standards, two design guidelines, namely “Concrete Society – Technical Report No. 55 Design Guidelines for Strengthening Concrete Structures using Fibre Composite Materials, Second Edition” and “The International Federation for Structural Concrete – Bulletin 14 Externally Bonded FRP Reinforcement for RC Structures” are adopted. As a conservative approach, the FRP design is checked against these two design guidelines so that requirements from both guidelines are satisfied. It was calculated that, after strengthening, the tensile stress of the concrete structure would be the allowable limit of 2.4MPa.
The adopted FRP material was a carbon fibre reinforced polymer plate of 1.4mm thick. The tensile strength of the material was 2,510MPa with a Modulus of Elasticity of 139GPa.

Quality control and durability of the materials are two main concerns during the approval process, especially the FRP material since this material is the first time being approved by the Buildings Department of Hong Kong. As parts of the approval conditions by Buildings Department, a series of quality control tests on the FRP system is required, including tensile tests for tensile strength, pull off tests for adhesion, tapping tests for void identification, verification load test and structural health monitoring, etc. Among these tests, verification load test and structural health monitoring are discussed in more details in the following text.

**SYSTEM INSTALLATION AND VERIFICATION LOAD TESTS**

The installation of strengthening systems started in 2008 and was completed in July 2009. The installation was in accordance with the following procedure. References were made to “Concrete Society – Technical Report No. 55 Design Guidelines for Strengthening Concrete Structures using Fibre Composite Materials, Second Edition” and “Concrete Society – Technical Report No. 57 Strengthening Concrete Structures using Fibre Composite Materials: Acceptance, Inspection and Monitoring”. Some photos showing different steps of installation and the verification load tests are also given.

(i) Conduct verification load test before strengthening systems’ installation
(ii) Conduct tensile test for FRP material as quality control
(iii) Construct anchorage blocks for prestressing bars, which were reinforced concrete blocks attached to the existing bridge structure.
(iv) Install prestressing bars and stress them in stages
(v) Install corrosion protection system to prestressing bars
(vi) Install CFRP plates on bridge soffit (Figure 1).
(vii) Conduct quality control tests to CFRP plates, including pull off tests, tapping tests, etc.
(viii) Conduct verification load test after strengthening systems’ installation (Figure 2 & 3)
(ix) Install optical strain gauge system
(x) Apply a proprietary fire rated fire protection layer to CFRP plates
(xi) Completion of works (Figure 4)

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![Figure 1: Prestressing bar and CFRP plate installation](image1)

![Figure 2: Lorry for verification load tests](image2)

![Figure 3: Strain measurement during verification load tests](image3)

![Figure 4: Bridge after strengthening works](image4)
PERFORMANCE MONITORING

The method of FRP strengthening was the first time approved by the Buildings Department in Hong Kong. Due to the pioneer nature of the works, a comprehensive structural health monitoring programme is implemented to monitor the structural health of the bridge and the strengthening system, including visual inspection, pull off test and optical strain monitoring.

Visual inspection is carried out to the whole bridge soffit, including the external prestressing system and CFRP system, with the aid of binoculars if necessary. If defects are observed, such as spalls, cracks, signs of corrosion, they will be recorded.

Pull off tests are carried out to verify the bond strength of CFRP plates over time. In addition to the required CFRP plates in accordance with the design, extra CFRP plates are installed at the bridge soffit, known as the Sacrificial Zone. Pull off tests are conducted to the CFRP plates at the Sacrificial Zone at the required time. Bond strength of CFRP plates are then determined.

Pull-off tests were carried at Year 2 and Year 5, i.e. 2012 and 2015. The test locations are shown in Figure 5 below.

![Figure 5 Locations for pull-off test](image1)

Optical strain gauges are installed on the CFRP plates at six locations as required by the Buildings Department. The strain gauges’ locations are shown in Figure 6 below. Cables from sensors are routed to a termination box located at the soffit of the East Span. During a standard monitoring, a readout unit is connected to the sockets on the test station and send optical signals to the sensors for measurement. Monitoring results are displayed on the laptop computer connected to the readout unit. The data obtained are wavelengths in the unit of nano-metre (nm). With formulas and parameters from the optical strain gauge supplier, the measured wavelengths are then converted to change in strains.

![Figure 6 Locations of optical strain gauges](image2)
Selection of Strain Gauges

During the selection of strain monitoring method, several types of strain gauges were considered, including electrical resistance gauge, demountable mechanical gauge and optical strain gauge.

An electrical resistance gauge is in the form of a flat grid of wires mounted on a thin plastic sheet. Strain is measured by means of changes in electrical resistance resulting from extension and compression of the gauge. Electrical resistance gauge is the most common type of strain gauges. However, an electrical strain gauge is made of metal or alloy which is prone to corrosion. It is not suitable for the use in an exposed environment for a prolonged period. Therefore this type of strain gauge is not adopted for this project.

A demountable mechanical (demec) gauge consists of a sensitive dial gauge mounted on a steel bar with a pivoted pointer at one end. This gauge is used to measure the relative movement between two studs attached to the surface concerned. When there is a relative movement, the studs cause the pivoted pointer to rotate which turn causes the dial to change. This method would depend on the skills of the technicians since the readings would be affected by the force pressing onto the studs. This method also requires accesses to the bridge soffit which is not available. Therefore this method is not adopted.

One of the most common types of optical strain gauges is Fibre Bragg Grating (FBG) sensors. A FBG sensor consists of an optical fibre containing a grating. During measurement, a light source with a range of different wavelengths is sent in the fibre. The wavelength corresponding to the grating pitch is reflected by the grating while all other wavelengths pass through the grating undisturbed. Since the grating period is strain dependent, measuring the reflected wavelength is able to determine the strain. An optical fibre is made of quartz, which is a non-metal and does not have corrosion protection. This type of sensors is suitable for outdoor conditions and is more durable. Considering that the monitoring programme will last for ten year, this monitoring technique is adopted.

Monitoring Frequencies and Acceptance Criteria

The monitoring frequencies of the above monitoring methods are summarized in Table 1 below:-

<table>
<thead>
<tr>
<th>Item</th>
<th>Test Method</th>
<th>Frequency</th>
<th>Acceptance Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Visual inspection</td>
<td>Quarterly during Year 1 Bi-annually during Years 2 and 3 Annually during Years 4 to 10</td>
<td>No apparent visual defect</td>
</tr>
<tr>
<td>2</td>
<td>Pull off test</td>
<td>Two tests at Years 2, 5 and 10</td>
<td>Pull off strength not less than 1.5MPa with no sign of significant movement</td>
</tr>
<tr>
<td>3</td>
<td>Strain Monitoring</td>
<td>Six test areas with the same frequency as (1) above</td>
<td>Measured strain not exceeding 1.8%</td>
</tr>
</tbody>
</table>

Monitoring Results

Strengthening works of the bridge were completed in July 2010 and the structural health monitoring then started. Up to end of 2015, monitoring results of the first 5 years have been obtained.

Visual inspection showed that the conditions of the bridge soffit and the installed strengthening works appeared in good conditions. No signs of defects were observed during each visual inspection. Both the CFRP plates and the fireproofing system remained intact on bridge without any cracks, delamination or spalling.

Pull-off tests carried out at sacrificial zones in 2012 and 2015 are shown in Table 2. The average pull-off strength is 2.07 and 3.90MPa. Therefore, the results satisfied the 1.5MPa requirement by the Buildings Department. It indicated that, the adhesive of the CFRP was still in good condition without degradation after 5 years.

Optical strain monitoring has been conducted for five years. The changes in strains over time are shown in Figure 7 below. The changes in strains range between -0.048% and 0.005% and are well below the limit of 1.8%. Therefore, all visual inspection, pull-off test, and strain monitoring results show that the strengthened bridge is of good conditions.
Table 2  Summary of pull-off test results

<table>
<thead>
<tr>
<th></th>
<th>Year 2012</th>
<th>Year 2015</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st result</td>
<td>1.63 MPa</td>
<td>3.71 MPa</td>
</tr>
<tr>
<td>2nd result</td>
<td>2.51 MPa</td>
<td>4.08 MPa</td>
</tr>
<tr>
<td>Average</td>
<td>2.07 MPa</td>
<td>3.90 MPa</td>
</tr>
</tbody>
</table>

CONCLUSIONS

In this paper, a case study of extending a concrete bridge’s service life using FRP strengthening and external prestressing is presented. Structural health monitoring is being implemented to the strengthened bridge. Monitoring results for the first five years are obtained successfully. The results show that the strengthened bridge is in good condition. It is concluded that the adopted strengthening system is effective.

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Highways Department. Structures Design Manual for Highways and Railways, 2006, HKSAR
Mini-symposium on Near-Surface Mounted FRP for Structural Strengthening

Organizers:

Shishun ZHANG
Jose SENA-CRUZ
BOND ON NSM CFRP SYSTEMS: RECENT CONTRIBUTIONS OF UMINHO ON DURABILITY, QUALITY CONTROL AND DESIGN

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ABSTRACT
In the last years, significant research in the context of bond of near-surface mounted (NSM) fibre reinforced polymer (FRP) systems in concrete has been conducted at the Department of Civil Engineering of the University of Minho. This paper presents a brief summary of the major results obtained in that research, namely in terms of durability, quality control and design topics. Accelerated ageing tests on NSM FRP bond specimens were conducted to simulate different environmental conditions. A new method was developed and applied to investigate the evolution of the adhesive stiffness and the bond behaviour of NSM systems for different curing conditions used for quality control of FRP installations. Regarding the bond design, two existing guidelines’ formulas were adapted to the partial safety factors framework.

KEYWORDS
Concrete, NSM, CFRP, Bond, Durability, Design.

INTRODUCTION
The near-surface mounted technique (NSM) is one of the most effective techniques to strengthen concrete structures in flexure, which presents several advantages when compared to the externally bonded reinforcement (EBR) that preceded it. The use of fibre reinforced polymer (FRP) as reinforcing material in the context of the NSM technique has significantly evolved in the last decades, both in terms of scientific investigations and practical applications (Coelho et al. 2015; Sena-Cruz et al. 2015).

One of the most critical aspects regarding the NSM technique is related to the bond behaviour of the composite system, i.e. the stresses transfer between concrete and the FRP reinforcement. To better understand that behaviour, bond tests have been carried out worldwide. Despite the existence of a manifold of test setups, those can be grouped in two main types: (i) direct (DPT) and (ii) beam (BPT) pullout tests (Coelho et al. 2015). The effectiveness and reliability of FRP-based strengthening systems depends fundamentally on the bond between the composite material and the concrete substrate. In recent years, the application of thermosetting resins in civil engineering applications has largely increased, mainly for their use in structural strengthening systems such as FRP reinforcements. The most common resins employed as structural adhesive for bonding FRP to structural elements to be strengthened are two-component epoxy resins (ACI 2008; Coelho et al. 2015; Sena-Cruz et al. 2015).

The lack of a comprehensive, validated, and easily accessible database about the durability and long-term performance of FRP systems (such as NSM FRP and EBR FRP systems) used in civil infrastructure applications has been identified as a critical barrier to widespread acceptance of these systems/materials by structural designers and civil engineers (Al-Mahmoud et al. 2014). Furthermore, the importance of developing procedures for quality control of FRP reinforcements, installation procedures and strengthening systems is recognized by several design guidelines (e.g. FIB 2001; ACI 2008). The development of methodologies could be essential to the definition of monitoring protocols for FRP installation and it could allow to quantitatively evaluate of its installation. Despite the significant research that has been conducted in the structural behaviour of NSM as a strengthening technique, there is not enough knowledge about the durability and quality control of NSM FRP systems, a critical aspect that must be taken into account when designing a structural strengthening. With the aim of improving the knowledge on these relevant issues, experimental research was developed.
Finally, in the absence of design formulations to estimate the bond strength of NSM FRP systems based on the Eurocodes philosophy, a design proposal was developed based on existing guidelines that were adapted to the partial safety factors framework.

**BOND DURABILITY OF NSM CFRP SYSTEMS**

**Experimental Programs, Test configuration and Materials**

In order to contribute for bridging the gap on the knowledge on durability of bond behaviour of NSM CFRP-concrete systems, an extensive experimental program composed of 23 series (each one composed of 4 specimens) of DPT tests was carried out to study the effects of different environmental conditions. In addition, the mechanical characterization of the involved materials was also assessed over the time. The specimens were firstly exposed to different environmental conditions for different periods of time ranging between 4 and 24 months, and then they were monotonically tested up to failure. The code name given to each series follows the format “Xn” where “X” defines the environmental condition (LE – laboratory environment; TW – tap water immersion; CW – immersion in water with chlorides; WD – wet/dry cycles in water with chlorides; TCA – thermal cycles with temperatures ranged between −15 °C to +60 °C; TCB – thermal cycles with temperatures ranged between +20 °C to +80 °C; FT – freeze/thaw cycles; real environments: REA – airborne salt/Mediterranean environment; REB – temperate environment), and “n” indicates the number of days/cycles that the series was submitted to the environmental condition (120, 180, 240, 480, and 720 days). For the case of the series TCA, TCB, FT, REA and REB in addition to the aged series, corresponding reference (R) series were also tested at the same time. The DPT specimens (see Figure 1a) consisted on concrete cubic blocks with 200 mm of edge, where a CFRP laminate with a cross-sectional area of 10 × 1.4 mm² was installed in a pre-cut groove opened in the concrete cover. The depth and the width of the groove were, respectively, 15 and 5 mm. A constant bond length of 60 mm, filled with the epoxy adhesive was adopted. The monotonic DPT tests were undertaken under force control at a load rate of 0.013 kN/s up to 10 kN and then under displacement control by a LVDT in loaded end section, at a rate of 2 μm/s. Mechanical properties of the CFRP laminate and epoxy resin are determined by performing the tensile tests (TT), while the mechanical characterization of the concrete was assessed by means of compression tests. Detailed information about the configuration of the DPT test, tensile tests on CFRP and epoxy adhesive and compressive tests on concrete, specimen preparation and ageing test procedures can be found elsewhere (Fernandes 2016).

**Results and Discussion**

The results obtained in the compressive tests on concrete specimens indicated an average compressive strength in cylinders, $f_{cm}$ of 36.0 MPa, with a coefficient of variation (CoV) of 3.9%, and an average Young’s modulus of 28.4 GPa (CoV = 5.8%), at 28-days of concrete age. However, due to the inclusion of fly ash (40% of the total binder content) in the concrete composition, the $f_{cm}$ continues to evolve up to one year of age; and, a maximum compressive strength of about 52 MPa (CoV = 2.5%) was achieved. Table 1 summarises the main results obtained in durability tests on the used CFRP and epoxy adhesive and the NSM CFRP-concrete system. In this table the meaning of each entity is the following: $f_{t_{FRP}}$ is the tensile strength of the CFRP laminate; $f_{t_{ad}}$ is the tensile strength of the epoxy adhesive; $F_{l_{max}}$ is the maximum pullout force of the bond system; $s_{l_{max}}$ is the slip at the loaded end at $F_{l_{max}}$. Table 1 also provides information about the failure mode (FM) of the bond tested specimens. Based on the obtained results, the following major observations can be drawn: (i) in general, all the environmental conditions investigated did not cause remarkable changes on the concrete compressive strength, except for the thermal cycles TCB which lead to a maximum reduction of 15%; (ii) CFRP samples presented negligible losses on their tensile properties when exposed to different environmental conditions; (iii) on the epoxy adhesive an increase up to 58% and 33% on the mechanical properties (tensile strength and elastic modulus, respectively) was observed, when the specimens were exposed to thermal cycles, due to a post-curing phase which occurs when temperatures higher than the ones experienced at the first curing are achieved. Contrarily, a significant reduction on its mechanical properties (up to 38% and 47% for the tensile strength and elastic modulus, respectively) was verified when the epoxy samples were submitted to wet environments due to water absorption (water uptake – plasticization phenomenon); (iv) the maximum reduction of about 12% on bond strength of the system was verified for the real environmental conditions (REA and REB); (v) thermal cycles between −15 °C and +60 °C improved the bond behaviour, with a maximum increase of 8% on bond strength; (vi) the effect of the exposure time also played an important factor on the degradation of bond properties, being greater on the specimens that aged for longer periods. It is important to note that the strong reduction on mechanical properties of the epoxy resin verified due to effect of some environmental actions did not have the correspondence on the global bond response of the NSM CFRP-concrete system, as can be seen in Figure 1b. One of the reasons that can be justified is associated to the obtained FM on bond specimens. As the maximum pullout force is limited by the type of FM and since the failure occurs mainly by debonding at adhesive/laminate interface, it means that the weakest component is the bond at

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adhesive/laminate interface (adhesion strength). Thus, the tensile strength of the epoxy adhesive is not directly comparable with adhesion strength at the interface between adhesive and laminate.

Table 1 Main results obtained in environmental tests.

<table>
<thead>
<tr>
<th>Series</th>
<th>CFRP $f_{f,RP}$ [MPa]</th>
<th>Adhesive $f_{f,adh}$ [MPa]</th>
<th>NSM CFRP-concrete system $F_{lmax}$ [kN]</th>
<th>$s_{lmax}$ [mm]</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>LE0</td>
<td>22.0 (4.5%)</td>
<td>24.25 (1.6%)</td>
<td>0.55 (11.1%)</td>
<td>I-FA(3)*</td>
<td></td>
</tr>
<tr>
<td>TCA120R</td>
<td>-</td>
<td>28.24 (6.8%)</td>
<td>0.69 (21.4%)</td>
<td>I-FA(2)<em>; I-FA+CS(1)</em></td>
<td></td>
</tr>
<tr>
<td>TCA240R</td>
<td>-</td>
<td>27.48 (3.4%)</td>
<td>0.70 (2.7%)</td>
<td>I-FA(4)*</td>
<td></td>
</tr>
<tr>
<td>LE240</td>
<td>-</td>
<td>26.71 (3.2%)</td>
<td>0.70 (3.9%)</td>
<td>I-FA(4)*</td>
<td></td>
</tr>
<tr>
<td>FT120R</td>
<td>2648.26 (1.76%)</td>
<td>-</td>
<td>28.77 (3.3%)</td>
<td>I-FA(3)<em>; I-FA+CC(1)</em></td>
<td></td>
</tr>
<tr>
<td>FT240R</td>
<td>-</td>
<td>28.72 (4.5%)</td>
<td>0.58 (12.0%)</td>
<td>I-FA(2)<em>; I-FA+CC(1)</em></td>
<td></td>
</tr>
<tr>
<td>LE480</td>
<td>20.8 (2.2%)</td>
<td>26.72 (4.5%)</td>
<td>0.58 (12.0%)</td>
<td>I-FA(2)<em>; I-FA+CC(1)</em></td>
<td></td>
</tr>
<tr>
<td>TCB180R</td>
<td>-</td>
<td>28.59 (3.3%)</td>
<td>0.69 (6.2%)</td>
<td>I-FA(3)<em>; C-C(1)</em></td>
<td></td>
</tr>
<tr>
<td>REA720R</td>
<td>-</td>
<td>28.63 (1.9%)</td>
<td>0.59 (9.4%)</td>
<td>I-FA(4)*</td>
<td></td>
</tr>
<tr>
<td>REB720R</td>
<td>-</td>
<td>28.63 (1.9%)</td>
<td>0.59 (9.4%)</td>
<td>I-FA(4)*</td>
<td></td>
</tr>
<tr>
<td>TW240</td>
<td>2629.58 (1.48%)</td>
<td>13.6 (4.9%)</td>
<td>26.93 (0.5%)</td>
<td>I-FA+CS(1)<em>; C-C(1)</em>; I-FA+CC(1)*</td>
<td></td>
</tr>
<tr>
<td>TW480</td>
<td>2573.58 (2.46%)</td>
<td>13.0 (2.1%)</td>
<td>26.94 (1.2%)</td>
<td>I-FA(3)<em>; I-FA+CC(1)</em></td>
<td></td>
</tr>
<tr>
<td>CW240</td>
<td>2504.52 (2.13%)</td>
<td>15.3 (2.9%)</td>
<td>28.01 (3.9%)</td>
<td>I-FA+CC(3)<em>; I-FA(1)</em></td>
<td></td>
</tr>
<tr>
<td>CW480</td>
<td>2459.38 (1.31%)</td>
<td>15.0 (1.7%)</td>
<td>27.58 (3.7%)</td>
<td>I-FA(2)<em>; I-FA+CC(2)</em></td>
<td></td>
</tr>
<tr>
<td>WD240</td>
<td>2601.36 (1.12%)</td>
<td>16.6 (4.2%)</td>
<td>27.93 (3.6%)</td>
<td>I-FA+CS(1)<em>; C-C(1)</em>; I-FA+CC(1)*</td>
<td></td>
</tr>
<tr>
<td>WD480</td>
<td>2455.77 (2.34%)</td>
<td>16.5 (2.5%)</td>
<td>26.34 (3.0%)</td>
<td>I-FA(3)<em>; I-FA+CC(1)</em></td>
<td></td>
</tr>
<tr>
<td>TCA120</td>
<td>2809.86 (1.89%)</td>
<td>25.9 (4.0%)</td>
<td>29.88 (1.6%)</td>
<td>I-FA+CS(2)<em>; I-FA(2)</em></td>
<td></td>
</tr>
<tr>
<td>TCA240</td>
<td>2642.79 (3.04%)</td>
<td>27.3 (2.3%)</td>
<td>29.75 (1.9%)</td>
<td>I-FA+CC(2)<em>; I-FA(2)</em></td>
<td></td>
</tr>
<tr>
<td>TCB180</td>
<td>2636.06 (2.74%)</td>
<td>32.9 (2.3%)</td>
<td>28.64 (4.0%)</td>
<td>I-FA(4)*</td>
<td></td>
</tr>
<tr>
<td>FT120</td>
<td>2609.14 (1.37%)</td>
<td>18.6 (0.6%)</td>
<td>28.63 (1.7%)</td>
<td>I-FA+CS(3)<em>; I-FA+CC(2)</em>; I-FA+CS(1)*</td>
<td></td>
</tr>
<tr>
<td>FT240</td>
<td>2666.74 (1.82%)</td>
<td>17.2 (2.5%)</td>
<td>27.40 (5.2%)</td>
<td>I-AC+CS(3)<em>; I-FA+CC(1)</em></td>
<td></td>
</tr>
<tr>
<td>REA720</td>
<td>-</td>
<td>-</td>
<td>25.34 (4.6%)</td>
<td>I-FA(3)<em>; I-FA+CC(1)</em></td>
<td></td>
</tr>
<tr>
<td>REB720</td>
<td>-</td>
<td>-</td>
<td>25.31 (1.0%)</td>
<td>I-FA(4)*</td>
<td></td>
</tr>
</tbody>
</table>

Notes: the values between parentheses are the corresponding coefficients of variation; *the value between parentheses is the number of specimens with this type of failure mode; I-FA = debonding at the interface FRP/adhesive; I-AC = debonding at the interface adhesive/concrete; C-C = cohesive shear debonding in concrete; CS = concrete splitting; CC = concrete cracking; AC = adhesive cracking.

The failure modes were classified with the generic denomination “X-Y”, where X defines the type of failure mode (interfacial - I or cohesive - C) and Y identifies the location where it occurred (concrete - C, adhesive - A, interface FRP/adhesive - FA or interface adhesive/concrete - AC). Besides the three main failure modes above described (I-FA, I-AC and C-C), in some specimens the final appearance also included one (or more) of the following damages (see Table 1): CS, CC and AC. The predominant failure mode occurred by I-FA (see Figure 1c). In fact, the pure interfacial failure is critical for FRP bars with a smooth surface since the smooth surface of CFRP strips used in this work is insufficient to provide mechanical interlocking between the laminate and the adhesive, and the rougher surface of the concrete leads to better bonding with the adhesive, the bond resistance relies primarily on chemical adhesion between the strip and the epoxy.

Figure 1 Durability tests: (a) DPT configuration; (b) Comparison between the variations of the $f_{f,adh}$ and $F_{lmax}$ due the distinct environmental conditions; (c) Main failure mode occurred in bond tests (I-FA).
QUALITY CONTROL OF NSM CFRP SYSTEMS

A new methodology for continuous monitoring of the evolution of elastic modulus of an epoxy adhesive used in FRP applications, based on adaptations of an existing technique originally devised for continuous monitoring of concrete elastic modulus since casting, called EMM-ARM (Elasticity Modulus Monitoring through Ambient Response Method) was proposed and validated (Granja et al. 2015). Afterwards, the influence of temperature on the curing process of the structural epoxy and its impact on the bond behaviour of NSM CFRP strengthening applications was investigated. For this purpose, an experimental program composed of three groups of tests, considering three different curing and testing temperatures (20 °C, 30 °C, and 40 °C) were developed:

(i) EMM-ARM tests (see Figure 2a) on adhesive samples to assess the evolution of the adhesive elastic modulus of the epoxy at different curing temperatures;

(ii) DPT tests on concrete cubic specimens strengthened with CFRP laminate strips, aimed at describing the development of the interface behaviour under variable curing conditions;

(iii) Tensile tests (TT) performed according to EN ISO 527-2:2012, to evaluate the elastic modulus value of the hardened epoxy.

The development of the epoxy elastic modulus obtained through EMM-ARM and the evolution of $F_{\text{limax}}$ obtained in bond NSM CFRP-concrete specimens along the curing time, at the three curing temperatures under test are presented in Figure 2b. The results of TT at 7-days of epoxy curing are also added to Figure 2b. The elastic modulus was calculated from TT results, according to the American Standard ASTM D638M-93. EMM-ARM applications on epoxy have demonstrated good repeatability of the experimental setup and procedures (Benedetti et al. 2016). The elastic modulus values estimated through the TT were lower than the values provided by EMM-ARM tests, with stiffness differences under 12.6% (~1.22 GPa for the 20 °C test). In addition, the results show that the reaction rates intensify with the increase of the curing temperature. For instance, the elastic modulus of 4 GPa is achieved at approximately 10.7 hours at 20 °C as opposed to the approximately 6.2 and 5.5 hours at 30 °C and 40 °C, respectively. These variations also occur in the duration of the dormant period, where adhesive stiffness is nearly null. With the increase in the curing temperature the duration of the dormant period becomes shorter, as can be observed in Figure 2b. At the reference temperature (20 °C) the setting time (herein defined as the time when the elastic modulus reached 0.25 GPa) is 4.5 ± 0.2 hours, as opposed to the shorter 2.6 hours observed in the test at 40 °C.

Figure 2b highlights that the peak pullout force and the epoxy elastic modulus obtained by EMM-ARM exhibit very similar evolution kinetics, thus indicating that the bond performance of NSM CFRP-concrete system strongly depends on the stiffness of the adhesive regardless of the curing temperature. The increase on bond stiffness is consistent with the stage at which the rate of thermosetting reactions is higher, although its development was slightly delayed compared to elastic modulus development. In general, $F_{\text{limax}}$ has a significant increase from 6 to 24 hours for the three analysed temperatures. For 20 °C, between 6 and 9 hours the peak pullout force increases by 3 kN and even by 10.56 kN in the subsequent 4 hours.

The slight difference on the kinetics of the two properties seems to be similar for all temperatures and may be attributed to a delay in the development of the molecular bond quality, which usually has less influence on the stiffness of the epoxy resin than on its strength (Moussa et al. 2012). Based on this kind of relationship, EMM-ARM can be employed for estimating the $F_{\text{limax}}$ and the minimum curing time to reach a threshold value of pullout force. In this manner it is possible to know the time required to put the strengthened structure in service, taking into account the influence of different environmental curing conditions.

![Figure 2](image_url)

Figure 2 Quality control tests: (a) Experimental setup of EMM-ARM; (b) Epoxy elastic modulus vs. peak pullout force along the curing time.
DESIGN OF BOND OF NSM CFRP SYSTEMS

In Coelho et al. (2015), the existing guidelines for the design and use of NSM FRP systems in concrete were analysed. At least four guidelines were identified: firstly, the CAN/CSA S6-06:2006 does not propose a closed-form formulation for evaluating the bond strength of NSM FRP systems. Alternatively, it refers that the bond strength should be obtained either by testing the NSM FRP system to be used, or it should be provided by the manufacturer; secondly the new annex of EN 1992-1-1:2013 is only applicable to FRP bars with rectangular cross-section (strips). In addition, its formulation requires some adhesive properties, such as tensile and compressive strengths, which are not often provided by the adhesives’ manufacturers. Furthermore, the formulation proposed by this guideline to estimate the bond strength depends on some coefficients which shall be provided by the manufacturer for each NSM FRP system, or adjusted by testing; the last two guidelines are the ACI 440.2R-08 (ACI 2008) and HB 305-2008 (SA 2008). These do not present the aforementioned drawbacks. In fact, they provide a set of closed-form expressions which are straightforward to apply since they depend on geometrical and mechanical parameters simple to obtain.

Hence, the ACI 440.2R-08 and HB 305-2008 guidelines were further analysed. Firstly, their accuracy was assessed based on a database with 363 and 68 direct and beam pullout tests, respectively (Coelho et al. 2015). The obtained results showed average errors of about 30% and 40% when ACI 440.2R-08 is applied to DPT and BPT, respectively; and 30% when HB 305-2008 is used for both DPT and BPT. These errors were computed by applying Eq. 1 to the total tests (N) in each database where, \( F_{f \text{max,Exp}} \) and \( F_{f \text{max,Num}} \) are the maximum pullout force values obtained in the experimental tests and by applying the corresponding guideline formulation, respectively.

\[
\sum_{i=1}^{N} \frac{|F_{f \text{max,Exp}} - F_{f \text{max,Num}}|}{F_{f \text{max,Exp}}} / N
\]

Then, a reliability analysis was conducted in order to allow using these guidelines under the framework of the Eurocodes design philosophy (Coelho et al. 2016). Since the amount of tests using CFRP strips was larger than the other types of FRP fibres/cross-sections, it was decided to conduct this task only for these types of tests. Table 2 presents the final formu eagles of each guideline (including the corresponding partial safety factors) that can be applied to estimate the bond strength of NSM CFRP strips. As can be seen, both formulations are based on the assumption that a certain length is required to develop the entire strength of the NSM FRP system (development length, \( L_d \)). If the bonded length, \( L_b \), is lower than \( L_d \), the maximum pullout force, \( F_{f \text{max}} \), will be linearly reduced according to \( L_b/L_d \). In Table 2 the following parameters are involved: \( A_t \), \( p_t \), \( E_p \), \( f_{k,c} \) and \( \gamma_t = 1.4 \) (FRP area, perimeter, elasticity modulus, characteristic tensile strength and partial safety factor), \( b_d \) and \( d_g \) (groove width and depth), \( f_{k,c} \) and \( \gamma_e = 1.5 \) (concrete characteristic compressive strength and partial safety factor), \( \tau_e \) and \( \delta_e \) (design bond strength and slip), \( L_{per} \) and \( \phi_{per} \) (failure perimeter length and ratio), \( \eta_p \) and \( \eta_b \) (safety factors). Detailed description and values of all parameters can be obtained in (Coelho et al. 2016).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>ACI 440.2R-08</th>
<th>HB 305-2008</th>
</tr>
</thead>
<tbody>
<tr>
<td>Development length ([L_d])</td>
<td>( \frac{A_t f_k}{\gamma_t} \left[ p_t \tau_e \right] )</td>
<td>( \frac{\pi}{2} \sqrt{\frac{\tau_e L_{per}}{\delta_e (EA)_t}} )</td>
</tr>
<tr>
<td>Maximum pullout force ([F_{f \text{max,d}}])</td>
<td>( \begin{cases} A_t f_k \sqrt{\gamma_t} &amp; \text{if } L_b \geq L_d \ A_t f_k \sqrt{\gamma_t} \frac{L_b}{\gamma_t} &amp; \text{if } L_b &lt; L_d \end{cases} )</td>
<td>( \begin{cases} \eta_p \sqrt{\tau_e \delta_e L_{per} (EA)<em>t} &amp; \leq A_t f_k \sqrt{\gamma_t} &amp; \text{if } L_b \geq L_d \ \eta_b \sqrt{\tau_e \delta_e L</em>{per} (EA)_t} \frac{L_b}{\gamma_t} &amp; \leq A_t f_k \sqrt{\gamma_t} &amp; \text{if } L_b &lt; L_d \end{cases} )</td>
</tr>
<tr>
<td>Other relevant information</td>
<td>( \tau_e = 1.77 \text{ MPa} )</td>
<td>( \delta_e = \left[ 0.73 \phi_{per}^{0.8} \left( \frac{f_{k,c}}{\gamma_e} \right) \right] / \tau_e )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \phi_{per} = (d_e + 1) / (b_e + 2) )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( L_{per} = 2(d_e + 1) + b_e + 2 )</td>
</tr>
</tbody>
</table>
CONCLUSIONS

This paper has presented a summary of the research conducted at the Department of Civil Engineering of the University of Minho in the context of bond of FRP NSM systems in concrete. Important contributions were made namely in the topics of durability, quality control and design. The environmental conditions investigated, which were considered to be quite severe, did not lead to significant changes on global bond performance of the NSM CFRP-concrete strengthening system, with a maximum reduction of about 12% on bond strength occurred under real environmental actions. EMM-ARM has potential to be employed for in-situ monitoring of the hardening of an epoxy adhesive curing in un-controlled conditions used in FRP applications. The bond behaviour of NSM CFRP-concrete systems is totally governed by the state of hardening of the adhesive. The peak pullout force and the epoxy elastic modulus obtained by EMM-ARM exhibit very similar evolution kinetics, thus indicating that the bond performance of NSM CFRP system strongly depends on the stiffness of the adhesive regardless of the curing temperature. Regarding the bond design, the major contribution consisted of adapting two existing guidelines’ formulas (American and Australian) to the partial safety factors framework (European philosophy). However, these formulas are not yet sufficiently accurate, thus further work should be carried out in this aspect and a newer and more accurate formulas should be developed.

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REFERENCES

FATIGUE BEHAVIOR OF NSM CFRP-STRENGTHENED REINFORCED CONCRETE BEAMS

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ABSTRACT

In this study, a series of reinforced concrete beams strengthened with near-surface mounted (NSM) carbon fiber reinforced polymer (CFRP) rods and strips were tested under four-point bending fatigue loading. Monotonic loading was applied to their corresponding control specimens to obtain the static monotonic flexural strength. Fatigue loading with a range of 10% of the corresponding static strength to various upper limits was applied to determine the empirical relationship for each type of specimen. It was found that the dominating failure mode under fatigue was rebar rupture. Mixed failure modes of rebar rupture and unstable cracks in concrete were also observed in specimens with longer fatigue life. As the specimens underwent more fatigue cycles, increase was observed in their tensile strains of CFRP and deflections due to accumulated damage and loss of stiffness.

KEYWORDS

Fatigue, Reinforced concrete (RC), Fiber-reinforced polymer (FRP), Near-surface mounted (NSM).

INTRODUCTION

The durability of many existing concrete structures is being compromised by corrosion, overload and material deterioration, and the structures are therefore in urgent need of rehabilitation (ASCE 2013). Compared to the conventional rehabilitation techniques and materials, fiber reinforced polymer (FRP) composites using near-surface mounted (NSM) reinforcements have appeared to be an attractive solution due to their ease of installation, high load-carrying capacity and flexural stiffness, and improved structural ductility (Hassan and Rizkalla 2003; Barros et al. 2006). For the overall flexural fatigue behavior of NSM-strengthened RC beams, several experimental studies exist in the literature. In general, NSM technique is reported to deliver great performance in aspects including longer fatigue life (Wahab et al. 2011), fewer signs of debonding (Quattlebaum et al. 2005), and lower stress ratio in prestressing strand (Rosenboom and Rizkalla 2006). Specimens were reported to fail by various modes, such as concrete crushing (Rosenboom and Rizkalla 2006), FRP debonding (Wahab et al. 2011) and rebar rupture (Badawi and Soudki 2009). Yost et al. (2007) concluded in their study that the addition of CFRP rod and strip also slightly increased the yielding load, while the ultimate static strength remained unchanged. Oadah and El-Hacha (2012) found the damage level of the strengthened structure was independent of the prestressing level of the NSM CFRP strip. Fernandes et al. (2015) concluded that the smooth NSM FRP strip did not affect either the ultimate load-carrying capacity or the failure mode of NSM-strengthened concrete slabs.

However, the recorded results are primarily driven by targeted performance comparison or improvement, in which case important fatigue-specific behavior, such as strain redistribution among steel, FRP and concrete, the deflection and crack progression throughout the fatigue life, and how those affect and lead to the final failure modes, oftentimes is either overlooked or lacking of detailed explanation. Moreover, these existing test data are still far from sufficient to develop accurate analytical models that are capable of predicting the physical fatigue behaviour and fatigue life of NSM-strengthened RC beams. In consideration of these research insufficiencies, this paper discusses a comprehensive experimental study of NSM CFRP rod- and strip-strengthened RC beams subject to both monotonic and fatigue loading under various load ranges. It includes important static behavior, fatigue life up to failure, and detailed fatigue flexural behavior between the uses of two different types of NSM reinforcement (e.g., strain redistribution, deflection, crack progression and stiffness degradation).
EXPERIMENTAL PROGRAM

Materials

The average 28-day and day-of-test compressive strength of concrete was experimentally obtained as 32.2 and 35.0 MPa, respectively, per ASTM C39/39M (ASTM 2005). Two types of CFRP reinforcement were used: CFRP round sand-coated spirally wound rod (R) and CFRP rectangular roughened strip (S), as listed in Table 1. Following ACI 440.3R-12 (ACI 2012), 3 standard material coupons for each type were prepared and tested to obtain the average mechanical properties of these reinforcements. All coupons failed due to rupture of CFRP reinforcement during the tests. Table 1 summarizes these experimentally obtained average material properties in comparison with the manufacturer’s data. Epoxy-based adhesive (S32 Hi-Mod) was used based on the results from a previous study that examined different types of adhesives (Lee et al. 2013). The bond strength, tensile and shear strength of the epoxy adhesive is 15.17, 48.26 and 42.95 MPa, respectively.

<table>
<thead>
<tr>
<th>Label</th>
<th>Type</th>
<th>Sizea (mm)</th>
<th>Section area (mm²)b</th>
<th>Perimeter-to-area ratioc</th>
<th>Surface conditiond</th>
<th>Elastic modulus (GPa)</th>
<th>Tensile strength (MPa)</th>
<th>Ultimate strain (%)</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>Round rod</td>
<td>10</td>
<td>71.26</td>
<td>0.44</td>
<td>SWSC</td>
<td>124</td>
<td>2172</td>
<td>1.75</td>
<td>MFR</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>79.9</td>
<td>0.39</td>
<td></td>
<td>147±6</td>
<td>2903±146</td>
<td>1.98±0.14</td>
<td>LABg</td>
</tr>
<tr>
<td>S</td>
<td>Rectangular strip</td>
<td>4.5×16</td>
<td>71.26</td>
<td>0.58</td>
<td>Ro</td>
<td>124</td>
<td>2172</td>
<td>1.75</td>
<td>MFRf</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>79.5</td>
<td>0.52</td>
<td></td>
<td>180±10</td>
<td>3413±117</td>
<td>1.91±0.16</td>
<td>LABg</td>
</tr>
</tbody>
</table>

*diameter for circular rod, thickness by width for rectangular strip; bsectional area is experimentally obtained by immersion testing (water displacement method); cPerimeter-to-area ratio = perimeter/sectional area; dSWSC = spirally wound and sand coated, Ro = roughened; eAverage with standard deviation; fManufacturer’s data; gLaboratory testing based on three identical FRP coupons.

Specimens

Twenty RC beams strengthened with NSM CFRP were cast in 2 batches, with a total length of 3,861 m and a rectangular cross-section of 203 mm wide and 305 mm deep (Figure 1). Each beam had two No. 6 rebars with a diameter of 19.05 mm as the tensile steel reinforcement at the bottom of the section, and two No. 3 rebars with a diameter of 9.53 mm as the compressive steel reinforcement at the top. Additionally, No. 3 U-shape shear stirrups were used at a space of 76.2 mm for the shear reinforcement. The concrete cover for the tensile and compressive steel reinforcement was 40 and 30 mm, respectively. Among the 20 beams, half was strengthened with NSM CFRP rods and the other half with NSM CFRP strips. Groove dimensions of 15 mm (width) and 25 mm (depth) were selected for both the CFRP rod and strip. The grooves were first cleaned by pressure washer and dried out by pressured air. The bottom half of the groove was then filled with epoxy resin followed by the embedment of the NSM reinforcements in the center and finished with filling up the top half of the groove. The grooves were allowed to cure for at least 14 days to develop the full strength in epoxy prior to any testing.

Each set of the specimens consisted of two identical specimens. The labeling scheme for the specimens (Table 2) was based on three parameters: the shape of the NSM reinforcement, the upper limit of the applied fatigue load, and the specimen number in the set. For instance, R-S-1 represents the first specimen in the set using NSM CFRP rod and subject to monotonic loading (static); S-75-2 represents the second specimen in the set with NSM CFRP strip and subject to a fatigue load ranging from 10% to 75% of its corresponding monotonic beam strength (Pu).
adaptation, during which load was applied in steel rebar slowed down [Figure 2(a)].

The compressive strain in the concrete [Figure 2(d)] existed but was less substantial.

Instrumentation and test setup

Each beam was instrumented with strain gauges and linear potentiometers, as shown in Figure 1. All beams were simply supported, and tested in four-point bending flexural condition. To better track and measure the crack length and crack opening width throughout the fatigue life, a small notch (10 mm deep) was pre-cut on the bottom surface of concrete at the mid-span to initiate the major flexural crack. This small notch was intended to facilitate the quantitative measurement of crack propagation throughout the test and did not affect the flexural behavior of the beams. Static monotonic tests were first conducted on control specimens and the other specimens in the same category were then tested under fatigue to a defined upper limit of the fatigue loading.

TEST RESULTS AND DISCUSSIONS

Monotonic test

Under the monotonic loading, the average monotonic beam strength ($P_m$) was recorded to be 158.06 and 145.45 kN (Table 2) for NSM rod and strip specimens, respectively. The two rod specimens and one strip specimen all failed by concrete crushing after steel rebar yielded (CC), and the other strip specimen (S-S-2) failed by interfacial debonding (D) between epoxy and CFRP reinforcement. Concrete started to crack at 13 kN and the yielding of steel rebar occurred when the load reached 110 kN. This can be confirmed from the sudden change of the slope in the load vs. strain and deflection curves as shown in Figure 2 for rod (R-S-2) and strip specimens (S-S-1). After yielding, the steel rebar experienced substantial increase in its tensile strain [Figure 2(a)] while load increased slowly. After the load reached approximately 130 kN, the tensile strain in steel rebar slowed down [Figure 2(a)] while the strain in FRP kept increasing steadily [Figure 2(c)]. This indicates a redistribution of load resistance beyond the 130 kN level between the NSM reinforcement and steel rebar, where strain hardening in the steel rebar only accounted for a very small stress increase. At the ultimate load level during the final stage (approximately 160 kN), most specimens failed by concrete crushing, and no failure was observed in the NSM reinforcements such as rupture or slip. Moreover, at the ultimate load level, the tensile strain of the steel rebar and FRP as well as the mid-span deflection were much greater in the rod specimen R-S-2 than those in the strip specimen S-S-1, implying a more ductile behavior in the specimens strengthened with NSM rods. The difference of the compressive strain in the concrete [Figure 2(d)] existed but was less substantial.

### Table 2 Summary of test results

<table>
<thead>
<tr>
<th>Specimen notation</th>
<th>Loading type</th>
<th>Fatigue load</th>
<th>Monotonic strength or fatigue (kN or cycles)</th>
<th>Stress range of steel rebar (MPa)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>R-S-1</td>
<td>Monotonic</td>
<td>NA$^b$</td>
<td>157.76</td>
<td>NA$^b$</td>
<td>CC</td>
</tr>
<tr>
<td>R-S-2</td>
<td>Monotonic</td>
<td>158.37</td>
<td>157.76</td>
<td>NA$^b$</td>
<td>CC</td>
</tr>
<tr>
<td>R-45-1</td>
<td>Fatigue</td>
<td>10-45</td>
<td>478.000</td>
<td>237</td>
<td>UC+RR</td>
</tr>
<tr>
<td>R-45-2</td>
<td>Fatigue</td>
<td>10-45</td>
<td>612.000</td>
<td>228</td>
<td>UC+RR</td>
</tr>
<tr>
<td>R-55-1</td>
<td>Fatigue</td>
<td>10-55</td>
<td>187.000</td>
<td>268</td>
<td>UC+RR</td>
</tr>
<tr>
<td>R-55-2</td>
<td>Fatigue</td>
<td>10-55</td>
<td>274.000</td>
<td>241</td>
<td>UC+RR</td>
</tr>
<tr>
<td>R-65-1</td>
<td>Fatigue</td>
<td>10-65</td>
<td>97.500</td>
<td>352</td>
<td>RR</td>
</tr>
<tr>
<td>R-65-2</td>
<td>Fatigue</td>
<td>10-65</td>
<td>80.200</td>
<td>337</td>
<td>RR</td>
</tr>
<tr>
<td>R-75-1</td>
<td>Fatigue</td>
<td>10-75</td>
<td>35.000</td>
<td>344</td>
<td>RR</td>
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<tr>
<td>R-75-2</td>
<td>Fatigue</td>
<td>10-75</td>
<td>42.000</td>
<td>404</td>
<td>RR</td>
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<tr>
<td>S-S-1</td>
<td>Monotonic</td>
<td>NA$^b$</td>
<td>159.31</td>
<td>NA$^b$</td>
<td>CC</td>
</tr>
<tr>
<td>S-S-2</td>
<td>Monotonic</td>
<td>131.59</td>
<td>159.31</td>
<td>D</td>
<td>D</td>
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<td>S-45-1</td>
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<td>10-45</td>
<td>1,480.000</td>
<td>186</td>
<td>UC+RR</td>
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<tr>
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<td>10-45</td>
<td>1,600.000</td>
<td>214</td>
<td>UC+RR</td>
</tr>
<tr>
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<td>410.000</td>
<td>236</td>
<td>RR</td>
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<tr>
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<td>393.000</td>
<td>245</td>
<td>UC+RR</td>
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<tr>
<td>S-65-1</td>
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<td>NA$^d$</td>
<td>NA$^d$</td>
<td>D</td>
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<tr>
<td>S-65-2</td>
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<td>187.000</td>
<td>265</td>
<td>UC+RR</td>
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<tr>
<td>S-75-1</td>
<td>Fatigue</td>
<td>10-75</td>
<td>84.200</td>
<td>348</td>
<td>RR</td>
</tr>
<tr>
<td>S-75-2</td>
<td>Fatigue</td>
<td>10-75</td>
<td>92.000</td>
<td>329</td>
<td>D</td>
</tr>
</tbody>
</table>

$^a$Percentage of monotonic strength of the respective group; $^b$Not applicable; $^c$Monotonic strength from static monotonic test or the number of cycles to failure from fatigue test (fatigue life); $^d$Not available since the specimen failed after the first cycle by insufficient curing of epoxy resin; $^e$Stress range of steel rebar after 1000 cycles of fatigue loading were applied; and $^f$CC = concrete crushing, UC = unstable crack, RR = rebar rupture, D = debonding.
Fatigue Test

**General Behavior** – Table 2 summarizes the main test results of the 16 strengthened beam specimens under fatigue loading varying from a fixed lower limit (i.e., 10% of the corresponding ultimate strength under monotonic loading, $P_u$) to a selected range of upper limit (i.e., 45%, 55%, 65% and 75% of $P_u$). The observed dominating failure mode was rebar rupture (RR) although a small group of specimens experienced some level of debonding in NSM that mostly affected their fatigue behavior. In some cases especially the beams subject to lower fatigue loading (with longer fatigue life), steel rebar rupture was occasionally accompanied with concrete crushing (UC+RR), which was resulted from excessive and unstable crack propagation in concrete. Only 1 out of 16 specimens failed due to premature debonding.

**Fatigue Damage** – Figure 3 displays the load vs. mid-span deflection response under different number of fatigue cycles for both types of specimen. It can be seen that the deflection is fairly linear during the loading and unloading throughout the fatigue life. A significant increase of the deflection was not observed until the final stage of the fatigue life was reached. The relation between the peak deflection at the mid-span and the normalized number of cycles with respect to its failure cycles is displayed in Figure 4. It can be seen that the progression of the peak deflection during the fatigue life is similar to the crack propagation behavior where three distinct stages existed. Such stage-wise response of peak deflection was mainly due to the progression of the fatigue cracks, the concrete fatigue creep, and the bond degradation of the NSM reinforcements. As can be seen from Figure 4, the deflections for beams under the 75%$P_u$ fatigue load (i.e., R-75 and S-75 series) are much larger than the others throughout the normalized fatigue life, while their corresponding number of fatigue cycle up to failure is the smallest among all. This could be due to the accumulation of large plastic deformation experienced in those specimens under high level of fatigue loading.

The stiffness degradation of both the rod and strip specimens throughout the fatigue life is illustrated in Figure 5. For both types of specimens, the stiffness degradation trends are similar among specimens under different levels of fatigue loading. It can also be seen that most of the degradation occurred during the first 5% of the fatigue life, Stage (1), after which the stiffness tended to stabilize showing slight degradation with constant rate within Stage (2). As the specimen approached its ultimate failure, Stage (3), the degradation became rapid again due to the large crack opening and propagation, and the shrinkage of effective area of the steel rebar. Similar trend was also reported elsewhere (e.g., Barnes and Mays 1999; Oudah and El-Hacha 2012).
**Figure 4** Progression of peak deflection at mid-span for: (a) rod specimens; and (b) strip specimens

**Figure 5** Stiffness degradation during the fatigue life for: (a) rod specimens; and (b) strip specimens

**Tensile Behavior of Steel Rebar and CFRP** – The response of load vs. tensile strain in steel rebar and NSM reinforcement under different number of fatigue cycles is shown in Figures 6(a) and 6(b) for rod specimens, and in Figures 6(d) and 6(e) for strip specimens. It can be seen that, as the number of fatigue cycles increased from 1000 (rod specimens) or 7000 (strip specimens) up to the prior-to-failure stage, the tensile strain curves for different number of cycles are close to each other, indicating a slow increase in the tensile strain in both steel rebar and NSM CFRP. At the prior-to-failure stage, the strengthened beams behaved differently as compared to during their earlier stages before the final fatigue life was reached. The tensile strain in steel rebar decreased considerably [Figures 6(a) and 6(d)], while the tensile strain in NSM reinforcement increased substantially [Figures 6(b) and 6(e)]. The compressive strain in concrete at mid-span [Figures 6(c) and 6(f)] generally had a small increase within the first two or three thousand cycles (e.g., from 857 to 1000 με in R-55-2 within the first 2000 cycles), which then remained constant throughout the rest of the fatigue life.

**Figure 6** Fatigue strains of reinforcements and concrete in specimens R-55-2 and S-55-1

**Fatigue Life** – Table 2 summarizes the number of cycles to failure (fatigue life) of each beam specimen. The fatigue load range (L) versus the number of cycles to failure curves is plotted in Figure 7. Based on the test results, two empirical equations are proposed to estimate the fatigue life of the strengthened concrete beams:

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\[ L = -25.47 \log(N) + 181.28 \quad \text{for rod specimens} \]  
\[ L = -25.44 \log(N) + 189.67 \quad \text{for strip specimens} \]

where \( L \) = fatigue load range and \( N \) = fatigue life. The R-squared value was 0.9760 and 0.9822 for the rod and strip cases, respectively. It can be seen that the NSM strip specimens generally have longer fatigue life than the rod specimens under the same load range (%\( P_u \)).

**CONCLUSIONS**

A total of twenty NSM CFRP strengthened reinforced concrete beams were tested to investigate their fatigue behavior. The main conclusions include: (1) A typical elastic-plastic behavior was observed for the strengthened specimens under monotonic loading with the “yielding” occurred when the tensile strain of steel reached 0.2%; (2) Most specimens under fatigue failed due to rebar rupture, while debonding and unstable crack was observed in specimens with lower fatigue load and longer fatigue life; (3) Deflection progression generally showed three distinct stages with the second stage counting for the majority of the fatigue life; (4) The internal bending moment contributed by steel and NSM generally remained unchanged, but the tensile strain drastically redistributed from steel to NSM during Stage (3); and (5) Empirical equations are proposed, where the NSM strip specimens showed longer fatigue life than the rod specimens under the same fatigue load range.

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BEHAVIOUR OF CONCRETE BRIDGE DECK SLABS STRENGTHENED WITH NSM FRP BARS

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ABSTRACT

Reinforced concrete bridge deck slabs receive traffic loads directly. Structural damage can occur, such as residual deformation and numerous cracks, which eventually decreases the life of the deck as well as its load carrying capacity. The use of near surface mounted (NSM) fibre reinforced polymer (FRP) is a promising strengthening technology for increasing the ultimate strength of reinforced concrete structures. However, the study of the behaviour of FRP NSM strengthening concrete bridge deck slabs is rather limited. This paper deals with strengthening reinforced concrete bridge deck slabs with NSM FRP bars. A series of one third scale BFRP retrofitted concrete deck slabs were conducted and tested. In the test, the supporting beam sizes and strengthening configuration were varied to investigate the behaviour of the FRP NSM strengthening test deck slabs. The test results show that FRP NSM technique improves the loading-bearing capacity of reinforced concrete deck slabs. The test on the deck system also presents that NSM BFRP retrofitting together with arching action had a cumulative beneficial effect on the loading-carrying capacity. Subsequently, a theoretical study was carried out and a model for prediction in loading-carrying capacities of NSM FRP strengthened concrete deck slabs was proposed. The ultimate strengths predicted by this theoretical method show good agreement with the test results.

KEYWORDS

Concrete bridge deck slab, NSM strengthening, FRP bars, Arching Action.

INTRODUCTION

Due to increasing the traffic load, structural damage can occur in reinforced concrete deck slabs, which eventually decreases the life of deck slabs (Takeshi 1994). For this reason, it appears to be significantly critical to strengthen the deteriorated concrete deck slabs. Nowadays, many different types of materials are used to strengthen deteriorated concrete members (Kumar et al. 1990). Due to high strength, stiffness-to-weight ratio and corrosion resistance, FRP laminates and strips have been widely used to retrofit or strengthen existing bridge structures using externally bonded (EB) technique (Teng et al. 2002). In the externally bonded FRP strengthened concrete deck slabs, FRP reinforcement could be highly susceptible to damage from collision, fire and temperature, ultraviolet rays and moisture absorption (De Lorenzis and Teng 2007). In some strengthened bridge cases, insufficient protection may reduce the service life of the structures. To overcome this disadvantage of externally bonded strengthening method and improve the utilization of the FRP materials, near surface mounted (NSM) FRP method was introduced as a promising technique (Zhang et al. 2013). Although there has been some research on NSM FRP strengthened concrete structures, the study of concrete slabs strengthened with NSM FRP reinforcement is rather limited, especially for strengthened concrete bridge deck slabs. Additionally, bridge deck slabs in the typical beam-and-slab type bridge have inherent strength due to in-plane forces set-up as a result of the restraint provided by the slab panel boundary conditions (Zheng et al. 2015). This is known as arching action (Zheng et al. 2010). This arching effect has a significant influence on the behaviour of concrete bridge deck slabs (Zheng et al. 2010, 2015). However, the effect of arching action on the behaviour of deck slabs strengthened with FRP has not been explored fully. Hence, this paper aims at investigating the combined effect of NSM FRP bars and arching action on the enhancement of the loading-carrying behaviour of strengthened concrete bridge deck slabs. In this study, a series of one-third scale concrete deck slabs strengthened by NSM FRP bars was conducted and tested in the laboratory. Thereafter, an understanding of the nature of behaviour of NSM FRP strengthened concrete bridge
deck slabs can be extended. Subsequently, the ultimate strengths of test specimens were predicted by the standard
design method (ACI Committee 440.2R-08, 2008) and the reported punching-strength theoretical method for FRP
strengthened concrete deck slabs (Hongseob and Sim 2004). The results indicated that those methods yielded
inaccurate predictions due to the neglected arching action. Based on the method of arching theory (Zheng et al.
2010), a modified model was proposed. The proposed approach was validated by the good correlation obtained
between the predicted results and the test results.

EXPERIMENTAL PROGRAMME

As shown in Figure 1, a one-third scale deck panel with dimension of 2000mm by 800mm was adopted to simulate
the behaviour of a real bridge deck supported by two steel I-section beams, which is determined based on the
previous study by authors (Zheng et al. 2014). The composite effect between the steel beam and concrete bridge
deck was achieved by the steel stud weld on the top flange of the steel beam. In this test, Basalt Fibre Reinforced
Polymer (BFRP) bars with a diameter of 6mm were used as NSM strengthening material. The mean tensile strength
and the modulus of elasticity of the tested BFRP bars were 1300 N/mm² and 68000 N/mm² respectively. Figure
2a illustrates the size and the location of the grooves, which were cut in the transverse direction at the bottom of
concrete deck slabs and cut to be 1.5 times of the diameter of NSM bars (De Lorenzis and Teng 2007). In the
strengthening procedure, the Sika (Sikadur-30) adhesive for bonding reinforcement was used as epoxy adhesive
with an average tensile strength of 46.8N/mm² and an elasticity modulus of 12000N/mm². In all the tests specimens,
the steel reinforcing bars with a diameter of 6mm were used as internal reinforcement. The spacing of steel bar
was configured to be 100mm.

Table 1 Nominal variables in experimental model

<table>
<thead>
<tr>
<th>Model</th>
<th>Strengthening Material</th>
<th>Configuration of NSM BFRP Bars</th>
<th>Reinforcement percentage</th>
<th>( f_{cu}^{**} ) N/mm²</th>
<th>Support Beam Size</th>
<th>Failure Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D-MBCon</td>
<td>N/A</td>
<td>-</td>
<td>0.5%</td>
<td>42</td>
<td>320×130×100mm</td>
<td>84</td>
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<tr>
<td>DS-SB1</td>
<td>BFRP bar</td>
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<td>0.5%</td>
<td>41.8</td>
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<tr>
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<td>0.5%</td>
<td>40.8</td>
<td>300×200×8mm</td>
<td>105</td>
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<td>BFRP bar</td>
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<td>0.5%</td>
<td>42</td>
<td>320×130×100mm</td>
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<td>0.5%</td>
<td>41</td>
<td>320×130×100mm</td>
<td>103</td>
</tr>
<tr>
<td>DS-MB1EB</td>
<td>BFRP bar + CFRP Plates</td>
<td>6mm@100mm</td>
<td>0.5%</td>
<td>41</td>
<td>320×130×100mm</td>
<td>116</td>
</tr>
</tbody>
</table>

**.-Concrete strength is tested based on 100mm×100mm×100mm cube test.

As shown in Table 1, the structural variables were external restraint stiffness and strengthening configurations. To
achieve a noticeable arching effect in this test, the influence of external restraint stiffness was studied by varying
the major and minor axis I value—\( I_{xx} \) and \( I_{yy} \) value of edge beams. To study the effect of NSM strengthening
configurations, the spacing of NSM FRP bars were varied from 100mm to 200mm, as shown in Table 1. In addition,
to avoid the debonding failure between NSM FRP and concrete, a combined strengthening method of NSM FRP
bars in transverse direction and externally bonded (EB) FRP plate in longitude direction was used in this test, see Figure 2b.

EXPERIMENTAL RESULTS

The cracking pattern at the bottom face of the six test deck models at failure was shown in Figure 3. All tested slabs had almost similar cracking patterns. The first cracks appeared directly under the loaded area and were oriented in the longitudinal direction parallel to the supporting beams. As the applied load increased, subsequent cracks propagated in the radial direction away from the loaded area. Compared to the unstrengthened deck specimen, fewer cracks with narrower widths were observed on the underside of the strengthened deck slabs. This indicates that the propagation of the cracking at bottom face was limited by NSM FRP bars. In addition, by using the externally bonded (EB) strengthening method in the longitudinal direction, less cracking was observed in the deck slab coded as DS-MB1EB.

The responses of load vs. vertical deflection at midspan of deck specimens are illustrated at Figure 4. It can be seen that, for all the test specimens, the load vs. deflection responses were very similar, which was almost linear up to failure, before the applied load reached the cracking load (around 25-30kN). After this loading level, the structural stiffness of strengthened deck panels was improved and a delay in the stiffness degradation was evident by using the NSM strengthening materials. As shown in Figure 4, the structural behaviour of NSM FRP strengthened concrete deck slabs were enhanced by increasing the supporting beam sizes. This could be due to the enhancement of aching effect (Zheng et al. 2010). Interestingly, the spacing of NSM FRP bars could not affect the load-deflection response significantly as shown in Figure 4, which could be due to the contribution of arching action to structural behaviour. Figure 4 illustrated that the stiffness of the concrete deck slabs strengthened by the
combination of NSM and EB methods (the specimen coded as DS-MB1EB) was slightly higher than that strengthened by NSM methods (the specimen coded as DS-MB1), and the strength were increased by around 10% over that of specimen. This indicates that two-directional strengthening with NSM FRP bars and EB FRP plates is more effective than the transverse NSM strengthening scheme. This finding is similar to that reported in the previous study (Sim et al. 2006).

![Figure 4 Load vs. deflection response in concrete deck slabs](image4)

The load-strain relationship for transverse steel reinforcement is shown in Figure 5. It was found that the applied load corresponding to yielding strain was significantly enhanced by using the NSM strengthening method. Additionally, it can be found that the steel reinforcement yielded prior to the punching failure in all the test specimens except the model with strong lateral restraint stiffness (model coded as DS1-BB1), see Figure 5. It is could be summarised that the contribution of steel reinforcement was reduced by increasing the aching action. Figure 5 illustrates that the relationship of load vs. reinforcement strain before the load reached the yielding load was not affected by the variation of supporting beam sizes. Conversely, reducing the spacing of NSM FRP bars resulted in larger yielding load due to the improved flexural stiffness. By using the combination of NSM and EB strengthening, a delay of yielding in transverse reinforcement was expected by the increased strengthening efficiency. With the further applied load, the contribution of steel reinforcement to loading-carrying capacity was reduced by increasing the supporting beam sizes and strengthening percentages.

![Figure 5 Strain values of steel reinforcement in concrete deck slab vs. applied load](image5)

Figure 6 shows the response of strain in NSM FRP bars and the applied load. It was found that this structural response is similar in all the strengthened test deck slabs before the applied load reached the cracking load. As the load increased, a delay in the stiffness degeneration was evident in the response of strain in FRP bars and the applied load in all the strengthened deck slabs. This indicated that the contribution of NSM FRP bars to loading-carrying mechanism significantly after the occurrence of cracking. As expected, the strain value of NSM FRP bars in the deck slab with small supporting beam (specimen coded as DS-SB1) and small strengthening ratio (specimen coded as DS-MB2) are larger than those from other strengthened deck slabs. Additionally, increasing the lateral restraint stiffness resulted in the smaller strain of NSM FRP bars at failure load. After the test, it was found that the rupture of FRP bars occurred in the deck slabs with small supporting beams (coded as DS-SB1). This suggests that the strengthening percentage is insufficient for this deck specimen. Therefore, it can be summarised that the effect of strengthening percentage and arching action should be both considered in the structural design.
In order to investigate the efficiency of the strengthening method, the comparison of loading-carrying capacity in all the test deck slabs was shown in Table 1. It can be seen that the ultimate load of deck slabs was increased by 25% by using NSM FRP bars in transverse direction with comparison of the results from the specimens coded as D-MBCon and DS-MB1. Increasing the supporting beam sizes resulted in larger ultimate loads, which is the same as that is reported in the works of concrete deck slabs by authors (Zheng et al. 2010). This indicates that the arching action has similar effect on loading-carrying capacity of NSM strengthened concrete deck slabs. In addition, the variation of spacing of NSM FRP bars (strengthening percentage) could not affect the ultimate loads significantly, which could be caused by the increased contribution from arching action (Zheng and Xia 2015). The test results showed that NSM FRP strengthening method, together with in-plane restraint providing arching action, had a cumulative beneficial effect on the ultimate capacity.

PREDICTION METHOD FOR ULTIMATE STRENGTHS OF NSM FRP STRENGTHENED CONCRETE DECK SLABS

<table>
<thead>
<tr>
<th>Model</th>
<th>( P_t ) (kN)</th>
<th>( P_{ACI} ) (kN)</th>
<th>( P_{hongseob} ) (kN)</th>
<th>( P_p ) (kN)</th>
<th>( P_t/P_{ACI} )</th>
<th>( P_t/P_{hongseob} )</th>
<th>( P_t/P_p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>D-MBCon</td>
<td>84</td>
<td>14</td>
<td>34</td>
<td>82</td>
<td>6.00</td>
<td>2.51</td>
<td>1.02</td>
</tr>
<tr>
<td>DS-SB1</td>
<td>96</td>
<td>33</td>
<td>52</td>
<td>102</td>
<td>2.91</td>
<td>1.86</td>
<td>0.94</td>
</tr>
<tr>
<td>DS-MB1</td>
<td>105</td>
<td>33</td>
<td>52</td>
<td>109</td>
<td>3.18</td>
<td>2.03</td>
<td>0.96</td>
</tr>
<tr>
<td>DS-BB1</td>
<td>120</td>
<td>33</td>
<td>52</td>
<td>124</td>
<td>3.64</td>
<td>2.32</td>
<td>0.97</td>
</tr>
<tr>
<td>DS-MB2</td>
<td>103</td>
<td>22</td>
<td>52</td>
<td>97</td>
<td>4.68</td>
<td>1.99</td>
<td>1.06</td>
</tr>
<tr>
<td>DS-MB3</td>
<td>116</td>
<td>33</td>
<td>52</td>
<td>107</td>
<td>3.52</td>
<td>2.24</td>
<td>1.08</td>
</tr>
<tr>
<td>Average=</td>
<td>103.5</td>
<td>33.5</td>
<td>52</td>
<td>97.7</td>
<td>3.66</td>
<td>2.21</td>
<td>1.00</td>
</tr>
<tr>
<td>Standard deviation=</td>
<td>1.57</td>
<td>0.57</td>
<td>1.57</td>
<td>0.57</td>
<td>1.57</td>
<td>0.57</td>
<td>0.57</td>
</tr>
<tr>
<td>Coefficient of variation=</td>
<td>0.15</td>
<td>0.15</td>
<td>0.15</td>
<td>0.15</td>
<td>0.15</td>
<td>0.15</td>
<td>0.15</td>
</tr>
</tbody>
</table>

In this study, the flexural strength of strengthened test slabs was predicted by a design code named as ACI 440.2R-08(2008). On the hand, a prediction method by Hongseob and Sim (2004) for externally bonded strengthened concrete deck slabs was adopted to predict the punching shear strength of test specimens. The predicted strengths by the current theoretical models of ACI 440.2R-08(2008) and Hongseob and Sim (2004) were shown in Table 2. It was evident that those two theoretical models yielded highly conservative predicted ultimate strengths for strengthened concrete deck slabs and the reliability of those two predictions was not good. To accurately determine the ultimate capacity of strengthened concrete bridge deck slabs, a proposed theoretical method based on arching theory in the authors’ previous study (Zheng et al. 2010) was used. In this proposed method, the bending capacity and was modified to be predicted by ACI 440.2R-08(2008). In addition, the influence of NSM FRP bars was assumed to be a contribution to "virtual reinforcement percentage" (Zheng et al. 2010) in the prediction of shear punching capacity. The test to predict ultimate capacities using the proposed theoretical method are listed in Table 2. It can be noted that this method yielded accurate and reliable punching strengths of the tested NSM FRP strengthened concrete deck slabs. The ratio of \( P_t/P_p \) is 1.0 with a coefficient of variation of 6%.
CONCLUSIONS

To investigate the behaviour of concrete deck slabs strengthened by NSM FRP bars, a series of one third scale concrete deck specimens were conducted and tested up to failure in the laboratory. The test results indicate that the NSM strengthening scheme effectively increased the concrete deck slabs punching capacities and stiffness. Debonding of NSM FRP reinforcement did not occur in the test process until the failure. By using the combination of NSM and EB FRP strengthening schemes, debonding of NSM FRP reinforcement was avoided, and then the capacity and stiffness was increased. The study of this combined strengthening method should be carried out in the further research. The tests on the deck specimens showed that NSM FRP strengthening, together with in-plane restraint providing arching action, had a cumulative beneficial effect on the ultimate capacity. In addition, it was shown that the contribution of steel reinforcement to the loading carrying capacity was reduced by increasing the lateral restraint and using the NSM strengthening method. The contribution of NSM FRP reinforcement was varied by the lateral restraint stiffness after the applied load reached the 80% of failure load. The NSM FRP strengthening showed a relatively greater increase in capacity for slabs with a lower external in-plane restraint. Therefore, this suggests that the effect of arching action should be considered in the calculation of NSM strengthening percentage for laterally restrained concrete deck slabs. In the comparison between the predicted strengths from the theoretical from literatures (ACI 440.2R-08, 2008; Hongseob and Sim 2004), it was shown that those methods yielded conservative prediction predictions. Because the influence of arching action was not considered, increasing the supporting beam sizes resulted in higher underestimation of ultimate strengths. The proposed prediction method was modified based on the arching theory and combined with the modified bending capacity prediction method and an “equivalent” area of arching reinforcement percentage. This punching strength prediction model predicted the loading-carrying capacities of the test slabs with reasonable accuracy.

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REFERENCES


ABSTRACT

Near Surface Mounted (NSM) strengthening technique has been used in a sustainable way for retrofitting existing structures. This technique, which utilizes CFRP laminates inserted in the concrete cover, has been used due to the several advantages when compared with the technique based on the application of these reinforcing materials on the concrete surface (EBR technique). Although several studies have been developed on this topic in the recent past, open issues still deserve research, such as the influence of the adhesive type on the performance of the NSM-CFRP system. The present work details an experimental program carried out in order to assess the effect of using three adhesives with distinct mechanical properties on the bond behavior of the NSM-CFRP system, through direct pullout tests (DPT). Thus, the following variables were considered in the present study: (i) the type of adhesive; (ii) the cross-section of the laminate; and, (iii) the bond length. The experimental pullout force-slip responses were obtained and digital image correlation (DIC) was used for obtaining additional information about the bond mechanisms developed. In general, two of the three adhesives, with similar mechanical characteristics, provided essentially similar bond behavior, with high level of effectiveness, whereas the third adhesive, which had a much lower elastic modulus than the other two, provided the lowest effectiveness in terms of the investigated parameters.

KEYWORDS

NSM, CFRP, direct pullout test, bond, adhesive.

INTRODUCTION

Fibre reinforced polymers (FRP) have been extensively investigated for repairing and/or strengthening existing structures. These materials can be introduced in the concrete cover of the element to be strengthened through the near surface mounted (NSM) technique. An epoxy adhesive is commonly used to fix the CFRP laminate to concrete. This bonding agent plays a critical role on the composite performance of the system. Extensive research has been developed to assess the bond behaviour of this strengthening system using carbon fibre reinforced polymers (NSM CFRP system). According to Coelho et al. (2015) the performance of the NSM CFRP system depends mainly on the: (i) geometry of the groove and of the FRP; (ii) mechanical properties of the concrete; (iii) mechanical properties of the adhesive; (iv) FRP cross-section and its external surface; and, (v) surface roughness in the groove. Digital Image Correlation (DIC) is a method that allows to evaluate the displacement fields at the surface of a structural element, as well as to compute the deformation fields during the test. Essentially the method is based on comparing two consecutive images of the element surface, before and after its deformation, through the application of an appropriate correlation technique (Chu et al. 1985). More information about this technique can be found elsewhere (e.g., Pereira et al. 2012; Carloni and Subramaniam 2013). In the literature few investigations can be found dedicated to analyzing the influence of adhesive and cross-section geometry of the laminate on the bond behavior of NSM CFRP system, e.g. Macedo et al. (2008).

In this research the effect of three adhesives for fixing CFRP laminates to concrete substrate according to the NSM technique is analyzed, by means of direct pullout tests. The main motivation for testing two stiff adhesives and one of much lower stiffness lies on the reported advantage of using highly deformable (flexible) adhesives in external bonding (EB) of CFRP laminates to RC beams as strengthening (Derkowski et al. 2013). In this research the type geometry of the CFRP cross-section as well as the bond length were also analyzed and as study variables. The experimental program is detailed and the main results are presented and analyzed in the subsequent sections.
EXPERIMENTAL PROGRAM

Test program, specimens and test configuration

The experimental program was composed of 51 direct pullout tests (DPT) where the influence of adhesive type, cross-section geometry and bond length of the bond NSM CFRP system were analysed. Table 1 presents the experimental program which includes: (i) three adhesive types - Adhesive 1 (ADH1), 2 (ADH2) and 3 (ADH3); (ii) two cross-sections of CFRP laminate - 10×1.4 mm² (L10) and 20×1.4 mm² (L20); and, (iii) six bond lengths (Lb) - 50, 60, 80, 100, 200 and 300 mm. Concrete cubic specimens were adopted with Lb values up to 100 mm, while concrete prismatic specimens were adopted with Lb values of 200 and 300 mm. Each series was composed of 3 specimens, being its generic denomination ADHX_LYY_LbZZ, where X represents the adhesive type (1, 2 or 3), YY is the width of CFRP in millimeters and ZZ indicates the bond length also in millimeters.

<table>
<thead>
<tr>
<th>Type of adhesive</th>
<th>Type of specimen’s geometry</th>
<th>CFRP cross-section geometry, ( w_1 \times t_1 ) [mm²]</th>
<th>Bond length [mm]</th>
<th>Series</th>
</tr>
</thead>
<tbody>
<tr>
<td>Adhesive 1 (ADH1)</td>
<td>Cubic</td>
<td>10( \times ) 1.4 (L10)</td>
<td>60</td>
<td>ADH1_L10_Lb60</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>80</td>
<td>ADH1_L10_Lb80</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>100</td>
<td>ADH1_L10_Lb100</td>
</tr>
<tr>
<td></td>
<td>Cubic</td>
<td>20( \times ) 1.4 (L20)</td>
<td>80</td>
<td>ADH1_L20_Lb80</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>100</td>
<td>ADH1_L20_Lb100</td>
</tr>
<tr>
<td></td>
<td>Prismatic</td>
<td></td>
<td>200</td>
<td>ADH1_L20_Lb200</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>300</td>
<td>ADH1_L20_Lb300</td>
</tr>
<tr>
<td>Adhesive 2 (ADH2)</td>
<td>Cubic</td>
<td>20( \times ) 1.4 (L20)</td>
<td>80</td>
<td>ADH2_L20_Lb80</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>100</td>
<td>ADH2_L20_Lb100</td>
</tr>
<tr>
<td></td>
<td>Prismatic</td>
<td></td>
<td>200</td>
<td>ADH2_L20_Lb200</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>300</td>
<td>ADH2_L20_Lb300</td>
</tr>
<tr>
<td>Adhesive 3 (ADH3)</td>
<td>Cubic</td>
<td>10( \times ) 1.4 (L10)</td>
<td>50</td>
<td>ADH3_L10_Lb50</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>100</td>
<td>ADH3_L10_Lb100</td>
</tr>
<tr>
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<td></td>
<td></td>
<td>150</td>
<td>ADH3_L10_Lb150</td>
</tr>
<tr>
<td></td>
<td>Cubic</td>
<td>20( \times ) 1.4 (L20)</td>
<td>80</td>
<td>ADH3_L20_Lb80</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>100</td>
<td>ADH3_L20_Lb100</td>
</tr>
<tr>
<td></td>
<td>Prismatic</td>
<td></td>
<td>300</td>
<td>ADH3_L20_Lb300</td>
</tr>
</tbody>
</table>

The geometry of the pullout specimens and test configuration adopted for both types of geometries is presented in Figure 1. In the cubic specimen, which consisted on concrete cube blocks with 200 mm of edge, grooves were performed on their lateral faces with a cross-section geometry of 5×15 mm² in general, or 5×25 mm² for insertion of CFRP laminates L10 and L20, respectively. In order to avoid the premature specimen failure by the formation of a concrete fracture cone between the load end and the top of the block, 100 mm of unbounded zone was guaranteed between these two points (in the case of ADH3 series the distance adopted was 50 mm). A steel plate of 20 mm of thickness was used to fix the upper part of the concrete block to the stiff base through four M10 steel threaded rods, ensuring negligible vertical displacements. In the prismatic specimens, which consisted of concrete prisms of 150×150×600 mm³, only grooves with a cross-section of 5×25 mm² for insertion of laminates L20 were performed. The distance between the loaded end and the top of the prism was also equal 100 mm. The specimen was supported on a steel frame through a steel plate, threaded rods, horizontal bars and a hydraulic jack. The tests were performed under displacement control at the loaded end adopting two displacement rates: for stiff adhesives ADH1 and ADH2, 2 µm/s, and for flexible adhesive ADH3, 5 µm/s. A load cell, placed between the grip and the actuator, was used to measure the applied force, \( F \), and a linear variable displacement transducer - LVDT1 - was used to measure the loaded end slip (relative displacement between the concrete and the CFRP laminate at the loaded end).

In order to help with interpreting the evolution of the degradation mechanisms of the anchorage zone during testing, the surface of the specimens at which the laminates were inserted was analysed using a Digital Image Correlation procedure (Blaber et al., 2015). The evolution of the crack pattern was documented during the monotonic loading by processing a sequence of images with a constant time step. The lens used had an aperture of f11 and the focal length was 100 mm. Led lights were used to illuminate the surface of the specimen. The camera sensor was a full frame size, with 36 Mpix. Considering that the priority was to trace the initiation and propagation of the cracks during testing, the principal tensile strain fields were mapped considering a very fine facet mesh. This mapping was particularly important to identify the location of the first cracks with respect to the CFRP laminate load end and to document the process of initiation and propagation of new cracks during the entire loading sequence.
Materials characterization

The compressive strength of the concrete was assessed using cylinders with 150 mm of diameter and 300 mm of height, at 28 and 110 days after casting. The modulus of elasticity and the compressive strength were assessed according to LNEC E-397:1993:1993 and NP EN 12390-3:2009 recommendations, respectively. An average modulus of elasticity \( (E_{cm}) \) of 27.0 GPa, with a coefficient of variation, CoV, of 0.5% and an average compressive strength \( (f_{cm}) \) of 35.4 MPa (CoV = 4.8 %) were obtained at 28 days. At 110 days, \( E_{cm} = 28.3 \) GPa (CoV = 2.5%) and \( f_{cm} = 38.5 \) MPa (CoV = 2.1%) were obtained. The mechanical properties of the adhesives were assessed according to ISO 527-2:2012. The following average values were obtained for the elastic modulus \( (E_a) \) and tensile strength \( (f_a) \):

- ADH1 - \( E_a = 11.67 \) GPa (CoV = 0.51%) and \( f_a = 25.59 \) MPa (CoV = 7.40%);
- ADH2 - \( E_a = 7.57 \) GPa (CoV = 6.15%) and \( f_a = 17.19 \) MPa (CoV = 5.43%);
- ADH3 - \( E_a = 0.012 \) GPa (CoV = 9.09%) and \( f_a = 2.67 \) MPa (CoV = 12.49%).

The mechanical properties of CFRP laminates can be found in (Fernandes et al. 2015) for L10 and in (Sena-Cruz et al. 2013) for L20.

RESULTS AND DISCUSSIONS

Main results

Table 2 presents the main results obtained from the pullout tests, including the average results for each series, as well as the observed failure modes, including: \( F_{\text{max}} \) is the maximum pullout force reached during the test; \( \tau_{\text{max,avg}} \) is the average shear bond strength at laminate-adhesive interface, which was computed by dividing \( F_{\text{max}} \) by the contact area between the CFRP laminate and the adhesive, \( 2(w_f + t_f)L_b \); \( s_{\text{max}} \) is the slip at \( F_{\text{max}} \); \( \tau_{\text{max,avg}} \) is the average shear bond strength at laminate-adhesive interface, which was computed by dividing \( F_{\text{max}} \) by the contact area between the CFRP laminate and the adhesive, \( 2(w_f + t_f)L_b \); \( s_{\text{max}} \) is the slip at \( F_{\text{max}} \). The responses are similar to those obtained by e.g. Fernandes et al. (2015). They are mostly non-linear, probably as a result of the non-linear behaviour of the adhesive, as well as due to the debonding process. For the series ADH1 and ADH2, short post-peak branches are observed, related to the failure mode in the form of debonding at the laminate-stiff adhesive interface. On the other hand, the responses obtained for series ADH3 are characterized by the long post-peak branches, related to cohesive failure in the flexible adhesive. Comparing the response of series ADH1 and ADH2 with the response of ADH3, significantly higher ultimate loads \( F_{\text{max}} \) are obtained for the stiff adhesives and significantly higher slip \( s_{\text{max}} \) for the flexible adhesive. Simultaneously, ADH3 provides lower stiffness on the NSM CFRP system than ADH1 and ADH2, but higher ductility.

Failure modes

Failure modes obtained in ADH1 and ADH2 series include the debonding at the laminate-adhesive interface (see Figure 3(a) – microscope photography) and laminate failure (see Figure 3(b)). In series ADH3 the failure mode observed was a mixed failure mode, i.e., specimens failed due to debonding at laminate-adhesive interface in some parts of the bond length, as well as due to the cohesive failure of the adhesive close to laminate-adhesive interface on the other parts (see Figure 3(c)).
Influence of study variables on the bond behaviour

The influence of adhesive type and cross-section geometry of the CFRP laminate were assessed for different bond lengths, and the summary of the results is presented in Figure 4. As expected, and in accordance to the literature (e.g. Sena-Cruz 2005; Coelho et al. 2015; Peng et al. 2015), $F_{\text{max}}$ tends to increase with the increase of $L_b$, coinciding the upper limit of $F_{\text{max}}$ with the tensile strength of the CFRP laminate (e.g. ADH2_L10 series with $L_b$ of 80 and 100 mm). $F_{\text{max}}$ is higher for L20 series than L10 series, which probably is related to the higher contact area at both interfaces and the consequent superior capacity of stress transfer from the laminate to concrete. In

Table 2 Main results obtained from the pullout tests.

<table>
<thead>
<tr>
<th>Type of adhesive</th>
<th>Type of laminate</th>
<th>Series</th>
<th>$F_{\text{max}}$ [kN]</th>
<th>$\tau_{\text{max}, \text{avg}}$ [MPa]</th>
<th>$s_{\text{max}}$ [mm]</th>
<th>$F_{\text{max}}/f_{\text{tm}}$ [%]</th>
<th>FM</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ADH1</td>
<td>L10</td>
<td>ADH1_L10_Lb60</td>
<td>22.49 (1.5%)</td>
<td>16.44 (1.5%)</td>
<td>0.50 (13.8%)</td>
<td>60.77 (1.5%)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>ADH1_L10_Lb80</td>
<td>25.97 (2.1%)</td>
<td>14.24 (2.1%)</td>
<td>0.68 (3.3%)</td>
<td>70.20 (2.1%)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>ADH1_L10_Lb100</td>
<td>29.57 (3.4%)</td>
<td>12.97 (3.4%)</td>
<td>0.93 (7.1%)</td>
<td>79.92 (3.4%)</td>
</tr>
<tr>
<td></td>
<td>L20</td>
<td>ADH1_L20_Lb80</td>
<td>46.69 (4.5%)</td>
<td>13.63 (4.5%)</td>
<td>0.50 (7.0%)</td>
<td>58.36 (4.5%)</td>
<td>I-FA(2)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>ADH1_L20_Lb100</td>
<td>48.91 (4.1%)</td>
<td>11.43 (4.1%)</td>
<td>0.64 (7.1%)</td>
<td>61.14 (4.1%)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>ADH1_L20_Lb200</td>
<td>59.53 (3.0%)</td>
<td>6.95 (3.0%)</td>
<td>1.10 (22.7%)</td>
<td>74.41 (3.0%)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>ADH1_L20_Lb300</td>
<td>61.03 (2.6%)</td>
<td>4.75 (2.6%)</td>
<td>1.27 (17.2%)</td>
<td>76.28 (2.6%)</td>
</tr>
<tr>
<td></td>
<td>ADH2</td>
<td>L10</td>
<td>ADH2_L10_Lb60,1</td>
<td>24.25 (1.59%)</td>
<td>17.73 (1.59%)</td>
<td>0.55 (11.35%)</td>
<td>65.55 (1.59%)</td>
</tr>
<tr>
<td></td>
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<td>ADH2_L10_Lb80,1</td>
<td>36.52 (2.09%)</td>
<td>20.02 (2.09%)</td>
<td>0.88 (21.5%)</td>
<td>98.71 (2.09%)</td>
</tr>
<tr>
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<td>ADH2_L10_Lb100</td>
<td>35.60 (2.98%)</td>
<td>15.61 (2.98%)</td>
<td>0.81 (10.98%)</td>
<td>96.22 (2.98%)</td>
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<td>L20</td>
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<td>48.40 (4.6%)</td>
<td>14.13 (4.6%)</td>
<td>0.48 (29.0%)</td>
<td>60.50 (4.6%)</td>
<td>I-FA(3)</td>
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<td>ADH2_L20_Lb100</td>
<td>54.06 (4.4%)</td>
<td>12.63 (4.4%)</td>
<td>0.75 (11.9%)</td>
<td>67.57 (4.4%)</td>
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<td>ADH2_L20_Lb200</td>
<td>55.19 (6.4%)</td>
<td>6.45 (6.4%)</td>
<td>0.88 (10.0%)</td>
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<td>ADH2_L20_Lb300</td>
<td>60.36 (3.4%)</td>
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<td>2.01 (17.7%)</td>
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<tr>
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<td>2.35 (6.0%)</td>
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<td>1.12 (11.2%)</td>
<td>6.34 (6.0%)</td>
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<td>5.03 (6.9%)</td>
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<td>13.59 (6.9%)</td>
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<td>8.12 (6.3%)</td>
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<td>12.36 (0.5%)</td>
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<td>ADH3_L20_Lb300</td>
<td>28.57 (10.4%)</td>
<td>2.22 (10.4%)</td>
<td>2.71 (20.6%)</td>
<td>35.71 (10.4%)</td>
</tr>
</tbody>
</table>

Notes: the values between parentheses are the corresponding coefficients of variation (CoV); FM (Failure Modes): I-FA = Debonding failure at laminate-adhesive interface; C-A = Cohesive failure of adhesive close to laminate-adhesive interface; F = CFRP failure; the values between parentheses are the number of specimens where this failure occurred. ¹Results collected from publication Sena-Cruz et al. (2015).

Figure 2 Typical average pullout force vs. load end slip relationship, obtained in the experimental program for the same bond length of $L_b= 100$ mm and two CFRP width: (a) ADH1; (b) ADH2; (c) ADH3.

Figure 3 Failure modes observed in the experimental program: (a) and (b) ADH1 and ADH2; (c) ADH3.
general according to $F_{\text{max}}$, less stiff ADH2 is more efficient than stiffer ADH1, while ADH3 provides $F_{\text{max}}$ values significantly lower than the previous ones. For instance, in series L20 for Lb80 the $F_{\text{max}}$ obtained with ADH3 is only 12% of the average value obtained when ADH1 and ADH2 are used. However, the performance of ADH3 tends to be closer to series ADH1 and ADH2 with the increase of $L_b$ (see series L20 with Lb300).

$t_{\text{max,avg}}$ tends to decrease with the increase of $L_b$ for ADH1 and ADH2 (stiffer adhesives) due to the higher contact area between the CFRP and adhesive and the non-uniform distribution of bond stresses along the bond length, as referred and justified in (Coelho et al. 2015). Using ADH3 (flexible adhesive), it can be noted that $t_{\text{max,avg}}$ tends to be similar for all tested bond lengths, which can be justified by a better and more uniform distribution of bond stresses along $L_b$ due to the lower stiffness of the adhesive ADH3, which has an elasticity modulus three orders of magnitude lower than the ones of ADH1 and ADH2. The cross-section geometry of the laminate did not significantly influence $t_{\text{max,avg}}$, in the present work: $t_{\text{max,avg}}$ was fond to be similar for L10 and L20, for the tested bond lengths (except ADH2 series). This finding probably demonstrates that the bond stress development at laminate-adhesive interface is independent of the cross-section of the laminate. Finally, it can be noted that the adhesive type has a significant influence on $t_{\text{max,avg}}$. Series ADH2_L10 reached higher $t_{\text{max,avg}}$ values than series ADH1_L10, while on ADH1_L20 and ADH2_L20 the values obtained were similar in both cases. $t_{\text{max,avg}}$ on series ADH3 was significantly lower than in the case of both ADH1 and ADH2 series.

$s_{\text{max}}$ also tends to increase with the increase of $L_b$ (except between series ADH2_L10_Lb80 and ADH2_L10_Lb100, probably due to the laminate failure that took place for $L_b$ equal or higher than 80 mm). $s_{\text{max}}$ is also influenced by the cross-section geometry of the laminate. For instance, for $L_b$ of 80 and 100 mm with ADH1 and ADH2, $s_{\text{max}}$ tends to be higher for series L10, on contrary to ADH3 where it was higher for laminate L20, namely for $L_b$ of 100 mm. Finally, $s_{\text{max}}$ is higher for ADH3_L20_Lb300 series than for ADH1_L20_Lb300 and ADH2_L20_Lb300 series.

Figure 4 Influence of study variables on the (a) peak pullout force, (b) maximum average bond strength and (c) loaded end slip at maximum pullout force.

Digital Image Correlation Analysis

Figure 5 presents two typical cases where DIC method was applied in order to compare the field strains of stiff and flexible adhesives. Figure 5(a) and Figure 5(b) present the results obtained for specimen ADH1_L20_Lb100_1 (“_1” means the first on specimen of the series) and ADH3_L20_Lb100_1, respectively at peak pullout force. In the first case, diagonal concrete cracks appear, which are caused by the stress transfer from the laminate to the concrete and produces the typical “fish spine” crack pattern due to the resistant mechanisms developed by the system (Sena-Cruz 2005). The use of stiff adhesives tends to lead to the concentration of damage on the concrete surrounding the reinforcing region and at the adhesive-concrete interface, remaining the adhesive almost intact. In contrast, the use of flexible adhesives leads to the significant damage concentration at the adhesive only, remaining the other materials almost intact.

CONCLUSIONS

This paper presents an experimental research on the bond behaviour of NSM CFRP system and on the evaluation the influence of the following parameters: (i) bonding agents (adhesives) with different mechanical properties; (ii) CFRP cross-section geometry; and, (iii) bond length. This assessment was performed through direct pullout tests (DPT). Specimens failed either by debonding at laminate-adhesive interface or by laminate failure with stiff adhesives depending on the bond length. In specimens strengthened adopting the flexible adhesive, the failure occurred by debonding at laminate-adhesive interface at some parts of the bond length, together with the cohesive failure of adhesive close to the surface of laminate on the remaining parts. In general an increase of the maximum pullout force with the increase of the bond length observed, as well as the greater system efficiency on transferring
stress between the laminate and the concrete substrate when the stiff adhesives were used. As expected, the pullout force was higher when the larger cross-section geometry was adopted for the laminate. Moreover, the deformations reached at the same load levels when flexible adhesives were used were significantly higher than the ones obtained when using the stiff adhesives. In general the average bond strength tended to decrease with the increase of bond length when utilising stiff adhesives, remaining approximately constant when the flexible adhesive was used. The cross-section geometry did not significantly influence the average bond strength. Stiff adhesives provided higher values of the average bond strength than flexible adhesives. DIC method allowed to document the development of the bond mechanisms for both types of adhesives during the entire loading sequence. The use of stiff adhesives led to a failure/damage concentration mostly at the concrete substrate and at the laminate-adhesive interface. In contrast, when the softer adhesive was used the damage/failure was found to be in the adhesive.

![DIC results obtained for the specimens (principal tensile strains at peak pullout force): (a) ADH1_L20_Lb100_1 and (b) ADH3_L20_Lb100_1. Note: the strains are presented in absolute value.](image)

**ACKNOWLEDGMENTS**

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**REFERENCES**


EXPERIMENTAL INVESTIGATION OF RC SLABS STRENGTHENED WITH NSM CFRP SYSTEM SUBJECTED TO ELEVATED TEMPERATURES UP TO 80 °C

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1 ISISE, School of the Civil Engineering, The University of Minho, PT. Email: jsena@civil.uminho.pt

ABSTRACT

The application of carbon fibre-reinforced polymers (CFRP) according to the near-surface mounted (NSM) technique has proved to be one of the most effective systems to strengthen existing reinforced concrete (RC) members in flexure. In spite of that, there are many open issues that deserve investigation such as the effects of exposure to elevated temperatures on the flexural behaviour of RC slabs strengthened with NSM-CFRP systems. The present work aims to experimentally evaluate the mechanical performance of RC slabs strengthened with NSM-CFRP systems under elevated temperatures by using steady-state and transient heating situations combined with applied loads. The temperatures studied were: 20, 40, 50, 70 and 80 °C for the steady-state tests, and 20 and 80 °C for the case of transient tests. Deflections, strains, temperatures and loads were registered in all phases of the tests, in order to thoroughly analyse the response of the system in terms of the load-deflection curves, evolution of the strains of concrete, CFRP and bond stresses between epoxy adhesive and CFRP. The experimental results have shown that the RC slabs strengthened with NSM CFRP systems presented a slight decrease in the ultimate strength and a change on failure mode at the temperature of 80 °C only.

KEYWORDS

Concrete, NSM CFRP, RC slab, strengthening, elevated temperatures.

INTRODUCTION

In the last few decades, the application of near-surface mounted (NSM) technique by using carbon fibre reinforced polymers (CFRP) has been increasingly used in the strengthening of reinforced concrete (RC) structures. One of the key aspects in the structural strengthening performance of a system is its response to the environmental conditions. In the case of structures strengthened with epoxy adhesives, the environmental temperature plays an important role, limiting the application range of such adhesives due to the glass transition temperature ($T_g$). The $T_g$ indicates the transition of an epoxy from solid to a viscous state taking place over a certain temperature range (of about 10-20 °C) (Silva et al., 2016; Michels et al., 2015). Since the bridges can easily reach high temperatures (close to 80 °C) due to sealing layer and asphalt application for example (Silveira, 1996), the performance of structures strengthened with FRP materials bonded with epoxy adhesives should be evaluated.

Several authors have been studying the behaviour of NSM technique under elevated temperatures, such as the work developed by Burke et al. (2013). The beams, strengthened with NSM CFRP systems, were initially loaded up to sustained load (40% of CFRP ultimate tensile strain) at ambient temperature and then heated up to 100 °C and 200 °C, concluding the strengthening system was capable of withstanding over 40 and 30 minutes at 100 °C and 200 °C, respectively. The failure mode observed was debonding at the epoxy adhesive-concrete interface whilst the beams tested at ambient temperature failed by splitting bond of the NSM CFRP system in the concrete adjacent to the epoxy-concrete interface. Apparently, NSM CFRP strengthening systems at the conditions previously specified, are able to maintain their structural capacity for short term periods of exposure to temperatures higher than the $T_g$ of the epoxy adhesives used ($T_g$=69 °C).

Firmo et al. (2014a) performed an experimental program comprising double-lap shear tests with CFRP strips installed according to the NSM technique for temperatures in interface varying between 20 °C and 150 °C. The $T_g$ of the epoxy adhesive used was about 55 °C. In the experimental procedure adopted, the specimens were firstly heated up to a predefined temperature (20, 40, 55, 90, 120, and 150 °C), and then loaded up to failure. From the bond-slip curves obtained, a reduction on the stiffness and maximum bond strength of about 16% at 55 °C was observed. Also, when compared with the control specimen (at 20 °C), a significant bond strength loss was achieved.
for the test with temperatures much higher than the $T_g$ of the epoxy adhesive, (84%, 40% and 33%, respectively, at 55, 90, and 120 °C) (Firmo et al., 2014a; Firmo et al., 2014b).

The present study aims to evaluate the behaviour of the RC slabs strengthened with NSM CFRP system when submitted to elevated temperatures in order to better understand the behaviour of the reinforced system during the glass transition process of the epoxy adhesive. An experimental program was developed including a total of 9 slabs, in which two different types of tests were performed: (i) steady-state tests, and (ii) transient tests. In the steady-state tests, the slabs were heated up to a predefined temperature, and then they were monotonically tested up to failure. In the case of the transient tests, firstly the slabs were preloaded up to 2/3 of ultimate load, and then submitted to an increase of temperature. When the slabs reached the target temperature, the slabs were unloaded and then, monotonically tested up to the failure. The temperatures tested were above and below the $T_g$ of the epoxy adhesive used the present study.

**EXPERIMENTAL PROGRAM**

Protocols adopted for the tests at elevated temperatures

The experimental program is divided in two main groups: (i) the steady state tests (SS) and, (ii) the transient tests (TR). In the first group six slabs (SL) strengthened by the NSM CFRP technique were tested at different target temperatures, namely, 20, 40, 50, 70 and 80 °C. In the second group, one slab was tested at the target temperature 20 °C and the other two at target temperatures of 80 °C. As shown in Figure 1, in the steady state tests, the slabs were firstly heated up to the target temperature without any load application, and then, after reaching this pre-set temperature, they were monotonically tested up to failure. In the case of the transient tests, the two slabs were initially submitted to a constant load of 2/3 of ultimate load (applied in a quasi-static manner) and, then, keeping the load constant, they were submitted to an increasing the temperature up to the predefined value (see Figure 1). One of the slabs was exposed to the temperature at about 80 °C over a period that last 4h, another one a period of 12h, approximately. After this phase the slabs were unloaded and subsequently loaded until failure. Moreover, one control specimen (reference slab) was monotonically tested up to the failure under at temperature of 20 °C.

**Specimen’s geometry and test set up**

The tested slabs used in the present program are 2000 mm long, 300 mm wide and 80 mm thick. Figure 2a depicts the geometry of the cross-section, including the longitudinal reinforcement (4 bars of 6 mm of diameter - 4 Ø6), corresponding to a longitudinal reinforcement ratio of 0.47%. The slabs were strengthened with three CFRP laminate strips, which corresponds an equivalent longitudinal reinforcement ratio of 0.68%. Figure 2b details the geometry of the groove and corresponding reinforcement.

Figure 1 Protocol of the test steady-state and transient tests

Figure 2 Slab’s geometry: (a) cross-section (b) details of the strengthening. [Note: all units are in millimetres]
A four-point bending test configuration was used, with 600 mm of shear spans. The deflections were measured through 5 LVDTs (see Figure 3a). A total of 7 strain gauges (TML BFLA-5-3-3L) were used to measure the strains developed on the CFRP laminates and steel bars, 5 strain gauges in sections S2, S3, S4, S5 and S6 (see Figure 3a) and 2 placed in S6, respectively. One strain gauge (TML PFL-30-11-3L) was used to measure the strain at the top layer of concrete at mid-span section. Finally, the applied load (F) was registered by a load cell placed between the grip and the actuator, with a static load carrying capacity of 200 kN (linearity error < ±0.05% F.S.).

The monotonic tests up to failure were performed by displacement control of 0.02 mm/s. For the case of the TR tests, the load of 2/3 of slab’s ultimate capacity carrying capacity, was applied under force control with a speed of 14 N/s. The unloading phase of unloaded at a velocity of 0.1 mm/s.

Figure 3b shows the general view of the prefabricated chamber and heating system where the SS and TR tests were performed, under controlled environmental temperature. The chamber was developed with extruded polystyrene foam and it was prepared to work up to 100 °C. Two industrial hot-air blowers heated the environment inside the chamber, controlled by an Arduino UNO (Arduino) using an MAX31855 thermocouple amplifier ADC, with cold-junction compensation. Also, a total of 25 thermocouples type-K were installed in order to register the temperature inside the chamber and, at different points of interest inside the slab, as shown in Figure 2a. Some holes were made to place thermocouples in the centre of the slab and at the depth of the strengthening (see Figure 2a). The number of thermocouples positioned in the mid-span cross-section were replicated in another cross-section at about 5 cm from the support and before the extremity of CFRP laminate strip (S1 in Figure 3a). Note the heating velocity of the materials, mainly concrete, was only dependent by the heating power of 6 kW of the hot-air blowers and by the target temperature fixed in the thermostat.

Material Characterization
The tested specimens had approximately 3 years of age and the compressive strength of cylinders (150 mm × 300 mm) was assessed by means of compression tests, following NP EN 12390-3:2011. The Young’s modulus was determined according to LNEC E397-1993:1993. The average concrete compressive strength was 51.9 MPa (CoV=3.9%) and Young’s modulus was 28.8 GPa (CoV=1.5%). The tensile properties of the CFRP laminate strips were assessed according to ISO 527-5:1997, presenting a tensile strength of 2648.3 MPa (CoV=1.8%) and a Young’s modulus of 169.5 GPa (CoV=2.5%). The uniaxial tensile properties of hardened epoxy adhesive were evaluated according to ISO 527-2:1993, and the following average values were obtained: 22 MPa (CoV=4.5%) for tensile strength, 7.2 GPa (CoV=3.7%) for Young’s modulus and 0.36% (CoV=15.2%) for the strain at the peak stress. The glass transition temperature (T_g) of the epoxy adhesive used is 55 °C (Silva et al., 2016). The steel longitudinal reinforcement was evaluated according to NP EN 10002-1:1990, and the obtained average values of E-modulus, hardening modulus and ultimate strength were, respectively, 212.2 GPa (CoV=6.3%), 0.7 GPa (CoV=6.6%) and 733.0 MPa (CoV=1.0%).
RESULTS AND DISCUSSION

Steady-state tests

Figure 4a illustrates the evolution of the air temperature inside the chamber and the average temperature measured at top of slab (concrete surface) – TC1, in the epoxy (inside of groove) – TC4/TC8 – and in the concrete core – TC3 – for the slab SL2_SS_80 (see also Figure 2a). The average temperature was calculated considering the temperature measured by two thermocouples positioned at the two different cross-sections. Although the temperature target was distinct for each experimental test, all slabs tested presented similar temperature development: in which the temperature evolution in first phase was linear during approximately one hour, attaining of about 60% the target temperature. Regarding to the CFRP and concrete strain measured during the heating phase in the slab SL1_SS_80, Figure 4a shows the strain evolution caused by the effect of temperature increase in the slab: (i) in the first 2 hours, up to of about 60 °C of temperature, both strains at the CFRP and concrete increase to the expansion of the slab; (ii) then, the CFRP strain continues to increase and the concrete strain starts to decrease. This behavior it may be explained by the fact of during this phase the slab is submitted to the self-weight in addition to the steel devices used for the application of the load which causes deflection and the balance between the loss of stiffness of the epoxy and the axial stiffness of the concrete and CFRP reinforcement. However, this statement requires scientific support based on further works to be developed, mainly numerical modeling.

Figure 4b plots all the load-deflection curves \( F \)-\( \delta \) obtained in the monotonic tests up to the failure (see also Figure 1). Table 2 includes the values of notable points of total load-deflection curves, such as: midspan deflection and applied load for crack initiation \( (\delta_{cr}, F_{cr}) \), yield initiation of the longitudinal reinforcements \( (\delta_{y}, F_{y}) \), maximum load \( (\delta_{max}, F_{cr}) \), and CFRP and concrete ultimate strains \( (\varepsilon_{CFRP} \text{ and } \varepsilon_{conc}) \). The values of \( F_{cr} \), \( F_{y} \) and \( F_{cr} \), the variation of slabs submitted to the temperature and the reference one (SL1_SS-20) are also represented in Figure 5b. The slab SL1_SS_40 presented the highest ultimate carrying capacity, even comparing with the reference slab (SL1_SS-20). The third branch of the \( F \)-\( \delta \) curve of this slab had a higher slope than the remaining slabs. Although, the air temperature has been constant during the heating phase and in the monotonic test up to the failure, the better performance of slab SL1_SS_40 may be related with a post-curing phase occurred during heating phase since the epoxy adhesive had been submitted to a temperature higher than the ones during the first its cure (Moussa et al., 2012). The post-curing can increase the mechanical properties of the material due to an increase in chain branching and molecular strength. Additionally, the adhesion mechanisms between concrete and the epoxy as well as the epoxy and the CFRP may be also improved. Also, slab SL1_SS_50 was found to have a higher stiffness of the third branch, even though the ultimate capacity has been lower than the reference slab. In this case the slab was submitted to temperatures close to this trigger point \( T_{c} \) justifying this weaker behavior. The slabs submitted to a temperature of 70 °C or higher, presented a stiffness reduction in the first and third branches when comparing with the remaining ones. This behavior is related to the temperature effect on the epoxy adhesive since the temperatures studied in the present work marginally affect the response of concrete itself (Neville, 1995; Anderberg et al., 1976). Moreover, the ultimate load started to be affected comparing to the reference one. The decrease was approximately 9.0%, 11.5% and 12.8% for slab SL1_SS_70, SL1_SS_80 and SL2_SS_80, respectively.

Regarding to the strains, the slabs SL1_SS_20 and SL1_SS_50 attained the maximum concrete strains of about 0.4%, while the maximum CFRP strains were registered in slabs SL1_SS_20 and SL1_SS_40 (at about 1.4%). The slabs tested between 20 °C and 70 °C failed by concrete crushing, while the slabs tested at 80 °C failed by
Transient tests

Figure 5a plots the evolution of deformations for the predefined temperatures. The deformation of the reference slab (SL1_TR_20) was at about 4.3 mm after 9 hours, while for the case of slabs SL1_TR_80 and SL2_TR_80 the deflection reached 11 mm and 14 mm after 4 and 12.5 hours, respectively. The high deformation in the slabs submitted to the 80 °C can be related with the simultaneous effects of creep of concrete and the decrease of the stiffness of epoxy adhesive. It is difficult to quantify the contribution of each effect individually since the creep deformation is greatly influenced by the temperature and the stiffness of the epoxy decrease significantly with the increase of temperature (Silva et al., 2016). According to literature, the creep in the concrete at mean temperature of 40 °C is 25% higher than that at 20 °C (1992-1-1, 2004).

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<th>Fmax [kN]</th>
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</table>

Notes: (a) values reached at the pre-loading phase; (b) strains measured at point-load section; (c) yielding of the reinforcement after the submitted a sustained loading.

Figure 5 Experimental tests (a) loading and heating phase of TR tests; (b) variation of the notable points.

Thereafter, the slabs were unloaded before being submitted to the monotonic tests up to the failure. Figure 4b depicts the total load – mid-span deflection curves in which, as expected, a linear response up to the deformation reached in the heating phase was observed. Table 2 includes the values of notable points of these curves. The stiffness in the first linear branch of the slabs tested at higher temperatures was not affected when compared with the reference slab. The behavior in the branch after the yielding of the longitudinal reinforcements was also similar between the slabs submitted at the same conditions, but marginally stiffer than the reference one (SL1_TR_20). Regarding to the ultimate load, the slabs SL1_TR_80 and SL2_TR_80 only present a decrease of -1.3% and -3.2% when compared with the reference (SL1_TR_20). The slight difference between the ultimate load of the slabs SL1_TR_80 and SL2_TR_80 can be related with the period of time that slabs were submitted to the load conditions, that was of 4 and 12 hours, respectively. Regarding to the strains, the maximum CFRP strain attained by the slab SL1_TR_20 was 1.4% and the concrete one was at about 0.4% and, therefore, the failure mode occurred by concrete crushing. Since the remaining two slabs (SL1_TR_80 and SL2_TR_80) failed by cohesive failure at the epoxy, the CFRP strain only reached about 1.27% and 1.23% in the slab SL1_TR_80 and SL2_TR_80, respectively, approximately 12% less than the reference.

Figure 5b presents the evolution of cracking, yielding and ultimate load of all slabs tested. As can be observed, the slabs submitted at 80 °C under steady-state tests presented a higher decreased of ultimate load than the slabs...
submitted to the same temperature in the transient tests. Regarding to the yielding load, the TR tests presented higher values due to the cracking caused in the loading and heating phase. Moreover, the behavior of the slabs of TR tests presented a higher stiffness up to the yielding load than the remaining slabs, as can be seen in Figure 4b. In these tests, after the yielding of reinforcement, the laminate does not reach strains as high as the remaining slabs, perhaps associated to the residual deflection after the unloading and the damaged caused by the period of time that slabs were submitted to the loading and heating.

CONCLUSIONS

The main goal of the present work was to study the behaviour of reinforced concrete slabs strengthened with CFRP laminate according NSM technique when subjected to the elevated temperatures. For this purpose, two different types of experimental tests were performed, namely: (i) steady-state (SS) tests, and (ii) transient (TR) tests. In the case of SS tests, the slab submitted to 40 °C presented the highest ultimate load comparing with the reference slab tested at 20 °C, that could be related with a post-curing occurred in the epoxy adhesive. The ultimate load of slabs submitted to 80 °C were the most affected by the temperature, and when compared with the reference one, a decrease of approximately 12% was observed. Regarding to the TR tests, the slabs submitted at high temperatures suffer 3 times more deformation than slab tested at 20 °C during the heating phase, that may be related with the decrease of the stiffness of epoxy and the increase of creep effect of concrete at higher temperatures. However, in the monotonic test up to the failure a maximum decrease of about 3% was observed in comparison to the reference. In both types of tests, the slabs submitted at 80 °C of temperature failed by cohesive failure at the epoxy, while in remaining slabs the failure mode was concrete crushing. From the test results, it can be seen that the elevated temperatures up to 80 °C only have marginal effect of the the behaviour of RC slabs strengthened with NSM CFRP systems. However, further works (e.g. numerical modelling of the tests carried out) are necessary for a better and more in-depth understanding of the test results.

ACKNOWLEDGMENTS

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ASSESSMENT OF CURRENT GUIDELINE FORMULATIONS FOR FLEXURAL STRENGTHENING OF REINFORCED CONCRETE BEAMS USING NSM REINFORCEMENT

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ABSTRACT

Near surface mounted (NSM) fiber reinforced polymer (FRP) reinforcement represents a valid alternative to externally bonded (EB) FRP reinforcement for strengthening existing reinforced concrete (RC) elements. When the NSM technique is employed for flexural strengthening, FRP composites are embedded into grooves cut into the concrete cover and filled with an inorganic or organic binding agent. Although many studies on the bond behavior of NSM FRP composites can be found in the literature, limited work is available regarding analytical models for designing NSM strengthening of RC members. In this paper, a database of experimental tests of 155 RC beams strengthened in flexure using NSM reinforcement is collected from the literature. The experimental results are employed to assess the accuracy of the analytical provisions obtained following the American, English, Canadian, and Italian guidelines.

KEYWORDS

NSM, RC beams, strengthening, analytical formulation, assessment.

INTRODUCTION

In the last decades, fiber reinforced composite materials have gained increasing popularity in the civil engineering industry. Among them, fiber reinforced polymer (FRP) composites have been employed to strengthen existing reinforced concrete (RC) and masonry elements. FRP composite strips are largely applied to existing elements as externally bonded (EB) reinforcement (see e.g. CNR DT-200 R1/2013). A valid alternative to EB FRP systems is represented by near surface mounted (NSM) FRP systems, which consists in FRP bars (i.e. circular cross-section) or plates (i.e. prismatic cross-section) installed into grooves cut onto the element surface and put in place using an adhesive (ACI 440.2R-08). NSM reinforcement are usually comprised of glass, carbon, or aramid fibers and are put in place through inorganic (e.g. cement grout) or organic (e.g. epoxy resin) binding agents. Although NSM reinforcement cannot be used in concrete elements for which the depth of cover is low, it is indicated for application where the concrete surface is exposed to potential damage, is undulating, or a thin layer of poor concrete is present near the surface (TR 55 2012). Extensive research has been carried out to investigate the NSM-concrete bond behavior. However, limited work is available regarding analytical models for designing NSM strengthening of RC members.

In this paper, three analytical models for the estimation of the bending capacity of RC beams strengthened with NSM FRP composites (i.e. American ACI 440.2R-08, English TR 55 (2012), and Canadian CSA S806-12 guidelines) are analyzed. Furthermore, the procedure for EB reinforcement proposed by the Italian CNR DT-200 R1/2013 is extended to the case of NSM reinforcement. An experimental database comprised of 155 RC beams strengthened in flexure with NSM reinforcement was collected from the literature. Comparisons between the maximum bending moment computed by the analytical models and the corresponding experimentally measured value allow to assess the accuracy of each model. Although conservative, the analytical models considered are generally poorly accurate and further studies are needed to provide reliable and accurate procedures for designing flexural strengthening with NSM reinforcement.

NSM FLEXURAL STRENGTHENING ANALYTICAL MODELS

The analytical procedures proposed by the American, English, Canadian, and Italian guidelines are herein briefly recalled. For each procedure, failure of the FRP reinforcement is assumed to occur when the strain in the FRP is

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According to the American guideline ACI 440.2R-08 the maximum strain of the FRP reinforcement $\varepsilon_{fd}$ is:

$$\varepsilon_{fd} = 0.7\varepsilon_f^*$$  \hspace{1cm} (1)

where the design rupture strain of the FRP reinforcement $\varepsilon_f^*$ is:

$$\varepsilon_f^* = C_e \cdot \varepsilon_f$$  \hspace{1cm} (2)

where $\varepsilon_f^*$ is the ultimate rupture strain of the FRP reinforcement and $C_e$ is an environmental reduction factor, which was assumed equal to 1.0 in this study. Reliability of the FRP contribution to flexural strength should be addressed by applying an additional strength reduction factor $\Psi_f$. Since $\Psi_f$ reflects uncertainties inherent in FRP systems that are prevented in laboratory tests, it was assumed equal to 1.0 in this study.

According to the English guideline TR 55 (2012), NSM strengthening of RC beams should be designed against different failure modes. To avoid concrete cover separation, the strain in the FRP should be limited to:

$$\varepsilon_{fd} = \varepsilon_{lim} = 38 \frac{b}{n} \frac{f_{ck}}{E_f A_f} \leq \frac{\varepsilon_{fu}}{E_f A_f}$$  \hspace{1cm} (3)

where $b$ is the width of the concrete cross-section, $n$ is the number of NSM profiles provided, $f_{ck}$ is the concrete characteristic tensile strength, $E_f$ is the FRP modulus of elasticity, and $A_f$ is the area of a single NSM profile. Furthermore, to avoid concrete splitting failure the strain in the FRP reinforcement in the cross-section where the FRP is needed, i.e. where yielding of the steel reinforcement starts to occur, should be limited to the maximum ultimate anchorage strain $\varepsilon_{max}$:

$$\varepsilon_{fd} = \varepsilon_{max} = \begin{cases} 
10b_{notchperim} \frac{f_{ck}}{E_f A_f} & \text{for } l \geq l_{max} \\
10b_{notchperim} \frac{f_{ck}}{E_f A_f} \left[ \frac{l}{l_{max}} - \frac{2}{l_{max}} \right] & \text{for } l < l_{max} 
\end{cases}$$  \hspace{1cm} (4)

where $b_{notchperim}$ is the perimeter of notch, $l$ is the FRP anchorage length, and $l_{max}$ is the maximum anchorage length:

$$l_{max} = 0.135b_{notchperim} \frac{E_f A_f}{f_{ck}}$$  \hspace{1cm} (5)

To prevent failure in the adhesive layer, the strain in the FRP should be limited to:

$$\varepsilon_{fd} = \varepsilon_{ad} = \frac{0.3f_{at} \cdot b_{haperim} \cdot l}{E_f A_f}$$  \hspace{1cm} (6)

where $f_{at}$ is the adhesive tensile strength and $b_{haperim}$ is the perimeter of NSM profile. For the sake of simplicity, the NSM reinforcement was assumed to be designed against NSM-concrete and NSM-adhesive separation failure (TR 55 2012).

The Canadian standard CSA S806-12 does not provide specific formulations to prevent FRP debonding failure but simply limit the strain in the FRP reinforcement to $\varepsilon_{fd}=0.007$. Therefore, when employing the Canadian formulation, failure is assumed either when the strain in the compressed concrete is equal to 0.0035 or when the FRP strain is equal to 0.007.

The Italian guideline CNR-DT 200 R1/2013 does not provide a specific procedure for designing RC beams strengthened with NSM reinforcement. However, in this paper the procedure provided by CNR-DT 200 R1/2013 for EB reinforcement was extended to the case of NSM reinforcement to investigate the possibility of using the same approach for both EB and NSM reinforcements. As a first attempt, the thickness $t_f$ and width $b_f$ of an EB FRP strip were related to the geometrical properties of the NSM reinforcement as follows:

$$b_f = \begin{cases} 
\frac{\pi \cdot d}{2n} & \text{for NSM bars} \\
\left( b_n + 2h_n \right) \cdot n & \text{for NSM plates}
\end{cases}$$  \hspace{1cm} (7)
where $d_n$ is the diameter of the $n$-th NSM bar, and $b_n$ and $h_n$ are the width and thickness of the $n$-th NSM plates, respectively.

According to the Italian guideline, the strain in the FRP reinforcement should be limited to $\varepsilon_{\text{FRP}}$.

$$\varepsilon_{\text{FRP}} = \min \left\{ \frac{\eta_{\text{FRP}} \cdot \varepsilon_{\text{p}, \text{FRP}}}{\gamma_{\text{FRP}}}, \varepsilon_{\text{IC}} \right\}$$

where $\eta_{\text{FRP}}$ and $\gamma_{\text{FRP}}$ are safety factors assumed equal to 1.0 in this study, $\eta_{\text{FRP}}$ is the FRP characteristic failure strain, and $\varepsilon_{\text{IC}}$ is the FRP intermediate crack-induced (IC) debonding strain:

$$\varepsilon_{\text{IC}} = \frac{k_b}{\gamma_{\text{IC}}} \left( \frac{2 \cdot k_b \cdot k_G \cdot f_{cm} \cdot f_{cm}}{f_{dd} \cdot f_{dd} \cdot k_{cm} \cdot (s_n/a_{cm})} \right)$$

where $k_b=1.25$ for distributed loads and $k_b=1.00$ otherwise, $\gamma_{\text{IC}}$ and $FC$ are a safety factor and a confidence factor, respectively, assumed equal to 1.0 in this study, $k_G$ is a corrective factor whose mean value is 0.32 mm, $f_{cm}$ and $f_{dd}$ are the concrete mean compressive and tensile strength, respectively, and $k_b$ is a geometrical corrective factor:

$$k_b = \sqrt{\frac{2 \cdot b_f}{b_f + b_f}} \geq 1$$

To prevent end-plate (EP) debonding failure, the FRP reinforcement strain in the cross-section where yielding of the steel reinforcement starts to occur should be limited to:

$$\varepsilon_{\text{steel}} = \min \left\{ \frac{1}{\gamma_{\text{steel}}} \left( \sqrt{\frac{2 \cdot k_b \cdot k_G \cdot f_{cm} \cdot f_{cm}}{f_{dd} \cdot f_{dd} \cdot k_{cm} \cdot (s_n/a_{cm})}} \right), \varepsilon_{\text{FC}} \right\}$$

where the maximum anchorage length $l_{\text{max}}$ is:

$$l_{\text{max}} = \min \left\{ \frac{1}{\gamma_{\text{steel}}} \left( \sqrt{\frac{2 \cdot k_b \cdot k_G \cdot f_{cm} \cdot f_{cm}}{f_{dd} \cdot f_{dd} \cdot k_{cm} \cdot (s_n/a_{cm})}} \right), 200 \text{ mm} \right\}$$

where $k_G$ is a geometrical corrective factor whose mean value is 0.063 mm, and $s_n$ is the FRP-concrete interfacial debonding slip.

**EXPERIMENTAL DATABASE**


**ASSESSMENT OF THE ANALYTICAL MODELS CONSIDERED**

The accuracy of each analytical model considered was evaluated by comparing the maximum bending moment measured experimentally, $M_{exp}$, with the corresponding analytical maximum bending moment $M_a$. The analytical maximum bending moment was obtained by enforcing the equilibrium of the cross-section assuming elasto-plastic behavior of concrete and steel and linear-elastic behavior up to $\varepsilon_{\text{FRP}}$ of the FRP reinforcement. As indicated by the specific guideline, the concrete stress-strain behavior was described by the elasto-plastic model proposed by Hognestad (1951) when assessing the American and Canadian guidelines, and by the parabola-rectangle model proposed by EN 1992-1-1 (2004) when assessing the English and Italian guidelines. It should be noted that all
safety factors and partial coefficients provided by the models were assumed equal to 1.0. Furthermore, when values of the adhesive tensile strength $f_{ad}$ were not reported, Eq. (6) was not considered in assessing the TR 55 (2012) model.

The discrepancy between analytical results and experimental evidence was assessed by computing the average Avg and coefficient of variation CoV of the ratios between the $i$-th analytical and experimental maximum bending moment, $r_i = M_{th,i}/M_{exp,i}$. Furthermore, the accuracy of each analytical model was assessed by computing the coefficient of variation $CoV_{ref}$ with respect to the perfect match average ratio $Avg_{ref} = 1.0$:

$$CoV_{ref} = \sqrt{\frac{\sum_{i=1}^{N} (r_i - Avg_{ref})^2}{N}}$$

(14)

where $N$ is the number of tests considered, equal to 155 in this study.

RESULTS AND DISCUSSION

Figure 1 shows the comparison between analytical and experimental values of the maximum bending moment for the 155 beams included in the database. Values below the line $M_{th}/M_{exp} = 1.0$ are conservative since the estimated bending moment is lower than the corresponding experimentally measured value, whereas values above the line $M_{th}/M_{exp} = 1.0$ are non-conservative.

Figure 1 Comparison between analytical and experimental maximum bending moment values of the 155 beams included in the database.

Values of the average Avg, coefficient of variation CoV, and perfect match coefficient of variation $CoV_{ref}$ obtained are reported in Table 1 for each model considered. The results obtained show that the American ACI 440.2R-08 and Canadian CSA S806-12 models provided the most accurate results, whereas the less accurate estimations were obtained by extending the Italian CNR DT-200 R1/2013 EB reinforcement model to the case of NSM reinforcement. Although a specific formulation to prevent NSM reinforcement debonding failure is not provided, the Canadian CSA design procedure seems particularly attractive because provides accurate results with a simple procedure. Similarly, the American approach provided accurate results with a simple procedure, which appears more reliable than the Canadian approach because the maximum strain in the composite is related to the NSM reinforcement failure strain and it is not a fixed value. However, ACI 440.2R-08 does not directly take into account the effect of the FRP reinforcement anchorage length, which can lead to premature failure along the anchorage
length. The Italian approach generally provided conservative results, which are attributed to the fact that the procedure adopted was originally developed for EB reinforcement that usually fails due to debonding at strain values lower than those observed for NSM reinforcement (see e.g. ACI 440.2R-08). The English TR 55 (2012) approach, which was developed specifically for NSM reinforcement and imposes several different analyses, generally provided conservative results with limited accuracy.

<table>
<thead>
<tr>
<th>Model</th>
<th>Avg</th>
<th>CoV</th>
<th>CoV_ref</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI 440.2R-08</td>
<td>1.02</td>
<td>0.33</td>
<td>0.33</td>
</tr>
<tr>
<td>TR 55 (2012)</td>
<td>0.70</td>
<td>0.29</td>
<td>0.42</td>
</tr>
<tr>
<td>CSA S806-12</td>
<td>0.90</td>
<td>0.29</td>
<td>0.31</td>
</tr>
<tr>
<td>CNR DT-200 R1/2013</td>
<td>0.55</td>
<td>0.27</td>
<td>0.53</td>
</tr>
</tbody>
</table>

**CONCLUSIONS**

This paper presents the assessment of four analytical models for the estimation of the bending capacity of RC beams strengthened with NSM reinforcement. The NSM reinforcement design formulations proposed by the American ACI 440.2R-08, English TR 55 (2012), and Canadian CSA S806-12 guidelines were considered. Furthermore, the procedure for EB reinforcement proposed by the Italian CNR DT-200 R1/2013 was extended to the case of NSM reinforcement. An experimental database comprised of 155 RC beams tested in flexure by 28 working groups was collected from the literature. The accuracy of each analytical model was assessed by comparing values of the maximum bending moment obtained from the analytical formulations with corresponding values measured experimentally. The results obtained showed that the Canadian approach seems particularly attractive because provides accurate results with a simple procedure. Similarly, the American formulation provided accurate results with a simple procedure that appears to more reliable than the Canadian approach because relates the maximum strain in the composite reinforcement to the NSM reinforcement failure strain. The English procedure, which was developed specifically for NSM reinforcement, generally provided conservative results with limited accuracy. The Italian approach generally provided conservative results, which are attributed to the fact that the procedure adopted was originally developed for EB reinforcement that usually fails due to debonding at strain values lower than those observed for NSM reinforcement. Based on the results obtained, further studies seem to be necessary to provide reliable and accurate procedures for designing flexural strengthening with NSM reinforcement.

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FIRE RESISTANCE OF RC MEMBERS STRENGTHENED WITH NEAR-SURFACE-MOUNTED FRP BARS

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ABSTRACT

The near-surface-mounted (NSM) fiber-reinforced polymer (FRP) technique has many obvious advantages including good bonding efficiency, the effectiveness of strengthening and especially the fire resistance. These performances can be highly affected by filling materials in the groove (refer to as the groove-filling materials or GFMs). In this paper, the performance of FRP bars under different working conditions in the fire and the impact of different GFMs on the fire resistance of reinforced concrete (RC) beams strengthened with NSM FRP bars are investigated. Through experimental and numerical studies, it was found that the fire resistance of RC beams filled with fiber polymer mortar (FPM) was improved significantly, while was inferior to the control beams when the filling material was modified acrylic emulsion mortar (MAEM). At the late stage of the fire test, strengthened members with longer adhesive anchorage length could delay the stiffness degradation and have relatively higher fire resistance. These preliminary test results indicated that the fire resistance of RC members strengthened with NSM FRP bars was highly dependent on the types of the GFMs.

KEYWORDS

FRP bars; NSM; strengthening; Fire resistance; groove-filling materials

INTRODUCTION

There are many coupling factors affecting the performance of FRP materials under high temperatures (Bisby et al. 2005; Wang et al. 2007). But in-depth studies on the performance of FRP materials under high temperatures are very lacking. Therefore, systematical researches on properties of FRP under different high temperatures, fire conditions and the analysis of its performance decline rules are necessary.

In recent years, near-surface-mounted (NSM) FRP reinforcing technique has attracted increasing attentions as a promising alternative to externally bonded (EB) FRP reinforcing technique. NSM method has many obvious advantages over EB method (Hassan et al. 2003; De Lorenzis et al. 2007; Rein et al. 2007; Wu et al. 2007; Ceroni et al. 2012; Burke et al. 2013; Seo et al. 2013; Bianco et al. 2014), such as higher anchoring capacity, better bond characteristics and even higher fire resistance. But its effectiveness of strengthening is greatly affected by filling materials in the groove (refer to as the groove-filling materials or GFMs). Fire resistance of RC beams strengthened with NSM FRP bars using different GFMs under different working conditions was tested in this paper. Two kinds of GFMs (fiber polymer mortar and modified acrylic emulsion mortar) and different adhesive anchorage lengths (300 mm and 400 mm) were considered respectively.

Because few studies have been conducted on the influence of cracks on the temperature transfer in the inner concrete, there are no clear conclusions about this. Vejmelková et al. (2008) had conducted a research about the influence of micro-cracks. It was concluded that the generated inner micro-cracks aggravated the porosity under high temperatures, and thus led to the reduction in thermal conductivity. However, there are no studies on the influence of macro-cracks on fire resistance of structures. In order to study the influence of the macro-cracks on the temperature field, the distribution and development of the temperature of some specimens with pre-fabricated cracks were monitored and the mechanism is analysed.
**EXPERIMENTAL PROGRAM**

Five cement-mortar specimens with pre-fabricated cracks were made, and the crack widths are 0.3mm, 0.5mm, 1.0mm, 1.5mm and 2.0mm, respectively. FRP bars were installed in the mould before cement-mortar pouring. The high temperature furnace used in the experiment is shown in Figure 1. As shown in Figure 2, the size of the mortar specimens was 75mm x 75mm x 300mm. The cracks were made by placing steel sheet with different thickness at the middle of the mortar blocks.

The specimens are numbered as S-0.3, S-0.5, S-1.0, S-1.5 and S-2.0. Three thermocouples were arranged on every FRP bar for measuring temperature at different points, the distance between the measuring points is 70mm. As shown in Figure 3, the upper and lower measuring points are located at the interior mortar, and the middle point is located at the pre-fabricated crack. The internal temperatures of mortar specimens are measured by TDS-303. The heating rate was set to be 5 °C/min to avoid spalling of the mortar blocks, referred to the previous experience by the authors and other researchers. When the target temperature (400 °C) was reached, the furnace temperature was maintained for a heating period of 1 hour. The furnace temperature and the temperatures at FRP bar at the three points (two inside mortar and one at the crack) were monitored.

For NSM FRP bars, three sides of FRP are protected by concrete and only one side is exposed to fire temperature filed. The GFM have significant effects on the bond property of concrete-FRP and on the fire resistance of strengthened beams. Previous tests (Vejmelková et al. 2008; Zhang 2014) have been conducted on NSM FRP bars strengthened RC beams using organic binder with/without fire protection. From those tests, for the two types of GFMs—organic binder and inorganic polymer composites, the former properties are superior to the latter under the normal temperature but inferior to the latter under the high temperatures. So it is of a great significance to combine the advantages of organic binder and inorganic polymer composite. Based on the previous research results, this paper focuses on the fire resistance of RC beams strengthened by NSM technique using organic glue at the end of the members and inorganic polymer composite materials at the middle grooves of the members.

A total of seven beams, with a span of 3.6m and a cross section of 0.2m x 0.45m, were tested. Details on the configuration of these beams are listed in Table 1 and Figure 4. Two specimens were tested at room temperature to acquire the ultimate bearing capacity, and the other five specimens were heated according to ISO-834 heating curve under 50% of the acquired ultimate load at room temperature. Beam L1 and L3 are the ordinary RC beams, L2, L4, L5, L6 and L7 are NSM strengthened beams with CFRP bars. K-type thermocouples in L3 to L7 were embedded at the different locations pointed out via sign “×” and serial numbers.

As shown in Figure 5, steel wire meshes-reinforced mortar was selected to guarantee the end anchorage property of NSM FRP bars in the fire, inorganic materials were used to fill the grooves in the middle part. The measured
average cylinder compressive strength of the C30 concrete, fiber polymer mortar (FPM) and modified acrylic emulsion mortar (MAEM) were 33.0MPa, 52.4MPa and 41.9MPa, respectively. The tensile strength and elastic modulus of CFRP bars with nominal diameter of 11 mm were 2285MPa and 177GPa, respectively. The yield strength and elastic modulus of the steel bar were 438.3MPa and 210GPa, respectively.

<table>
<thead>
<tr>
<th>Beam</th>
<th>filling materials</th>
<th>Protective length</th>
<th>Constant load level</th>
<th>Temperature field</th>
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<tbody>
<tr>
<td>L1</td>
<td>-</td>
<td>-</td>
<td></td>
<td>room temperature</td>
</tr>
<tr>
<td>L2</td>
<td>FPM</td>
<td>400mm</td>
<td></td>
<td>room temperature</td>
</tr>
<tr>
<td>L3</td>
<td>FPM</td>
<td>400mm</td>
<td></td>
<td>fire</td>
</tr>
<tr>
<td>L4</td>
<td>FPM</td>
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<td>fire</td>
</tr>
<tr>
<td>L7</td>
<td>MAEM</td>
<td>400mm</td>
<td></td>
<td>fire</td>
</tr>
</tbody>
</table>

FPM = fiber polymer mortar; MAEM = modified acrylic emulsion mortar

**Table 1 The parameters of bending beams**

**TEST RESULTS AND DISCUSSION**

*The effect of cracks on temperature transferring*

The average temperatures of the two measuring points at the internal mortar of cement-mortar specimens are compared with that of the measuring point at the crack. The temperature-time curves are shown in Figure 6. Only the results of S-0.3, S-1.0 and S-2.0 are showed because of the same variation regularity of S-0.5 and S-0.3, and the nearly same regularity between S-1.0 and S-1.5.

As shown in figure 6, all the temperatures at crack and internal point showed rising trends with the increasing of furnace temperature, and each of the two temperature-time curves emerged an inflection point at around 80 minutes when the furnace temperature reached the constant temperature stage. It can also be found that the temperature curves at internal point and crack point are nearly consistent before the target temperature (400℃) was reached. However, after the target temperature 400℃ was reached inside the furnace and kept constant, the increasing speeds of temperature at the crack point were higher than the internal points. At the end of the test, the temperatures at the crack point of S-0.3, S-1.0 and S-2.0 were 7℃, 13℃ and 22℃ higher than the internal point, respectively.

The above phenomena indicate that the presence of crack does not accelerate the transfer of temperature from external to the internal at the initial heating process. In addition, it can also be concluded that although a wider crack will lead to a slightly higher temperature transfer speed at the constant temperature stage, 2.0 mm crack...
width was still not serious for temperature transferring when the environment temperature was below 400 ℃.

After exposure to the high temperature, the FRP bars were taken out, as shown in Figure 7. The softness and decomposition of the resin and the combustion of fiber of S-2.0 were more serious; FRP bars of S-1.5 was slightly loose and bulged locally; the fiber and resin of S-0.3~S-1.0 had no obvious damage at the crack site. This indicated that although the FRP bars at the crack site directly were exposed to the high temperature, the decomposition of the resin was limited under the function of oxygen insulation. As a consequence, it is reasonable to infer that the FRP bars could still have certain residual mechanical property if only fine and dense cracks appeared in FRP bars strengthened RC members.

![Figure 7 Surface configuration of FRP bars after high-temperature](image)

**Analysis of beam deflection**

the deflection curves of L3 to L7 at different times under fire recorded by LVDT's located along the beam are shown in Figure 8. For example, the L3-30 curve in the Figure 8a represents the deflection curve of L3 after 30 minutes. Compared with L3 to L5, within 30 minutes, the deflection values of L6 and L7 were very small because of the non-negligible expansion of concrete during the early heating stage. For L3 to L5 within 90 minutes of fire exposure, the deflection curves were nearly similar and stable, while the deflection of L6 and L7 were much larger. The reason was that the MAEM suffered serious degradation of mechanical performance in high temperatures. It was peeled off from the beam bottom, and then the exposed NSM FRP bars directly contacted with the fire, which leaded to the degradation of stiffness and strength of the strengthened RC beams.

![Figure 8 The deflection curves of L3 to L7 under different times](image)

**Fire resistance of NSM beams**

The failure modes of L3 to L7 exposed to fire were concrete crushing in compression zone as well. The fire resistance, mid-span deflection and temperatures of rebars at mid-span are listed in Table 2. From Table 2, L4 and L5 with FPM have better fire resistance than L3 and L6 to L7 with MAEM, which means that the fire resistance of beams can be significantly improved when appropriate inorganic materials were used as groove filler. When comparing L4 with L5 and L6 with L7, respectively, it can be inferred that a longer end anchorage length with steel wire meshes reinforced mortar can increase the fire resistance.

**The effect of crack-width control**

Fine and dense cracks were observed on the grooves as shown in Figure 9a, and the number of cracks in the FPM was more than that in the bottom and side concrete. The mortar was not significantly peeled off after being cooled...
for 120 minutes. Only the bottom surface of exposed CFRP bar became loose due to the high temperature carbonization while the other part was still dense due to the oxygen barrier protection of mortar, and it could still bear loads.

### Table 2 Measuring results of beams during fire

<table>
<thead>
<tr>
<th>Beam</th>
<th>Fire resistance /min</th>
<th>Mid-span deflection /mm</th>
<th>Temperature of rebars /℃</th>
<th>CFRP</th>
<th>steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>L3</td>
<td>96</td>
<td>25.46</td>
<td>461.2</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>L4</td>
<td>119</td>
<td>36.06</td>
<td>721.5</td>
<td>502.1</td>
<td>472.8</td>
</tr>
<tr>
<td>L5</td>
<td>126</td>
<td>37.94</td>
<td>789.4</td>
<td>472.8</td>
<td>—</td>
</tr>
<tr>
<td>L6</td>
<td>94</td>
<td>68.49</td>
<td>693.2</td>
<td>401.8</td>
<td>479.5</td>
</tr>
<tr>
<td>L7</td>
<td>101</td>
<td>62.21</td>
<td>609.9</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

(a) L4

![Surface condition of L4 after fire exposure](image1)

(b) L6

![Surface condition of L6 after fire exposure](image2)

The surface condition of L6 after fire exposure is shown in Figure 9b, several cracks went through to the top of the beam in the pure bending section, the mortar spalling was serious in the groove of beam bottom, there was a 400 mm bar bare section at the left side of beam and a 200 mm bar bare section in the beading-shearing section at the right side of beam. The CFRP bar in the bare section was seriously carbonized and its resin was basically volatilized. In the other groove, mortar has swelled with a larger gap, and it basically lost thermal-insulation protection effect. The MAEM after fire exposure has become loose and can be even squeezed into granular substances. The crack-widths of L4 and L6 were tested.

The largest crack width of L4 and L6 measured after fire exposure was about 1 mm in the groove. Considering the crack closure after unloading and referring to the relationship between the deflections before and after unloading, it was estimated that the crack width when nearly reaching the fire resistance generally exceeded 2 mm.

It can be seen that the fire resistance of a strengthened beam would be sufficiently improved if there are only fine and dense cracks at the beam bottom. For further study, the relationship of FRP residual mechanical performance and heating period as well as how to use the relationship to analyze fire resistance of beams are worth examining.

### CONCLUSIONS

In this paper, the influence of cracks on the temperature transfer was tested first, and then the performance of NSM CFRP bars-strengthened RC beams with two types of GFMs and different lengths of end anchorage under fire exposure were experimentally studied. The main conclusions based on the results are as following:

1. As the crack width is limit, it will hinder heat transfer into the internal specimen, so the temperature of air at the crack is much lower than that in the high temperature field and is consistent with internal temperature. Meanwhile, while FRP material at the narrow crack site is in direct contact with air, its resin volatilization is less, so it can still maintain good mechanical properties. With the increase of the crack width, the hinder function gradually decreases and resin volatilization is more serious.

2. The adhesive anchorage length has little effect on the deflection in the early stage of the fire, but in the late stage of fire function, longer length of anchorage glue and protection can delay the decline of the stiffness of strengthened beam and its fire resistance is relatively high.

3. The performance of GFMs at high temperatures has great effect on the fire resistance of strengthened beam. The generated large crack width or spalling of the filled material can lead to direct exposure of FRP bars to
fire, which will cause a significant reduction of the fire resistance of the overall member.

(4) In the groove of the strengthened RC beam, the FPM produces fine and dense cracks under fire condition and is hard to peel off. By contrast, the MAEM is easy to burst, so it is important to select proper inorganic polymer mortar for meeting good fire resistance of NSM strengthening.

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INTERACTION FORCES IN RC BEAMS STRENGTHENED WITH NSM FRP ROUND BARS

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ABSTRACT
The near-surface mounted (NSM) strengthening method has attracted an increasing worldwide attention in the last decade. Although the bond efficiency between FRP and concrete in the NSM method is much improved compared with the externally bonded (EB) FRP strengthening method, debonding failures have also been often observed in reinforced concrete (RC) beams strengthened with NSM FRP bars. In such FRP-strengthened RC beams, debonding may initiate at either of the two ends of the NSM bar (i.e. end debonding), due to the existence of large localized interaction forces between the NSM bar and concrete near the bar ends. This paper presents an analytical solution to the interaction forces in RC beams strengthened with NSM FRP round bars, which are one of the most popular types of FRP bars used for NSM strengthening. The key elements of the proposed analytical solution are the two interfacial stiffness parameters (i.e. tangential interfacial stiffness and normal interfacial stiffness) and the eccentricity of the tangential interaction force to the centroid of the NSM bar. The accuracy of the analytical solution is verified with predictions from a sophisticated 3D finite-element (FE) model of a RC beam strengthened with a NSM round bar.

KEYWORDS
FRP, finite element (FE) analysis, interaction forces, interfacial stiffness, near-surface mounted (NSM), RC beams

INTRODUCTION
The use of near-surface mounted (NSM) FRP composites for strengthening RC beams in flexure has become increasingly popular in the last decade (Hassan and Rizkalla 2004; Sena Cruz et al. 2006; De Lorenzis and Teng 2007; Oehlers et al. 2008). In the NSM FRP method, grooves are cut into the concrete cover of a concrete member for the embedding of FRP bars using an adhesive. A typical schematic of NSM FRP for flexural strengthening of RC beams is shown in Figure 1. Compared to the externally bonded FRP method, the NSM FRP method has a number of advantages including a reduced risk of debonding failure and better protection of the FRP reinforcement (De Lorenzis and Teng 2007). However, the improved bond effectiveness cannot eliminate the possibility of debonding failure in RC beams strengthened with NSM FRP bars (Zhang et al. 2013; 2014). The debonding failure in an NSM FRP-strengthened beam may initiate from a major intermediate crack (i.e. IC debonding), or from either of the two ends of an NSM bar (i.e. end debonding) (Teng et al. 2006). For the latter case which has been more often observed, the failure are closely related to the existence of large localized interaction forces between the NSM bar and concrete near the ends of the bar in such strengthened RC beams.

Figure 1. Schematic of NSM FRP strengthening systems
Recently, Zhang and Teng (2013) proposed a closed-form solution to the interfacial interaction forces in RC beams strengthened with NSM rectangular FRP bars. Zhang and Teng's (2013) solution includes the establishment of approximate equations for the interfacial stiffness parameters, and its accuracy has been demonstrated by a 3D linear elastic finite element (FE) model. Following Zhang and Teng's (2013) work, this paper develops a closed-form analytical solution to the interaction forces in RC beams strengthened with NSM round FRP bars, with the key issue being on the establishment of interfacial stiffness parameters (Figure 2) and the eccentricity of tangential interaction force to the centroid of the NSM bar. In this paper, the governing equations and solutions proposed by Zhang and Teng (2013) are first presented, followed by determination of the two interfacial stiffness parameters and the eccentricity of tangential interaction force particularly for round NSM FRP bars. The proposed analytical solution is then verified by a sophisticated 3D FE model of RC beam strengthened with an NSM round bar.

GOVERNING EQUATIONS AND SOLUTION

The governing equations of interaction forces between NSM FRP and concrete (\( F_t \) and \( F_n \), i.e., the interaction forces in the tangential direction and the normal direction respectively) for RC beams strengthened with NSM bars subjected to a uniformly distributed load (UDL) \( q \) can be expressed as (Zhang and Teng 2013):

\[
\frac{d^2 F_t(x)}{dx^2} - k_T \left( \frac{d_s}{E_s I_s + E_f I_f} + \frac{1}{E_s A_b} \right) F_t(x) - k_F \left( \frac{d_s}{E_s I_b + E_f I_f} \right) V_r(x) = 0 \tag{1}
\]

\[
\frac{d^2 F_n(x)}{dx^2} + k_T \left( \frac{1}{E_s I_b} + \frac{1}{E_f I_f} \right) F_n(x) + k_f \left( \frac{y_b}{E_s I_b} - \frac{y_f}{E_f I_f} \right) \frac{dF_n(x)}{dx} + k_t \frac{1}{E_s I_b} q = 0 \tag{2}
\]

where \( x \) is the distance from the NSM bar end in the longitudinal direction; \( E_t \), \( A_b \) and \( I_f \) are the elastic modulus, cross-sectional area and second moment of area of the NSM FRP bar respectively; \( E_s \), \( A_b \) and \( I_b \) are the elastic modulus, cross-sectional area and second moment of area of the original beam respectively; \( E_n \).

\( G_n \), and \( t_n \) are the elastic modulus, shear modulus and thickness of the adhesive layer respectively; \( d_o \) is the distance between the centroids of the original beam and the NSM bar; \( y_b \) and \( y_f \) are the eccentricities of the tangential interaction forces to the centroid of the beam and that of the NSM bar respectively; and \( V_r \) is the shear force in the beam. \( k_T \) and \( k_f \) are the tangential interaction stiffness and the normal interaction stiffness respectively, which can be defined as the interfacial interaction forces between the NSM bar and concrete per unit length corresponding to a unit relative displacement between the NSM bar and concrete in the designated direction (as shown in Figure 2).

The solutions to Eqs. 1 and 2 are in the following form:

\[
F_t(x) = B_1 \cosh(\lambda x) + B_2 \sinh(\lambda x) + mV_r(x) \tag{3}
\]

\[
F_n(x) = e^{-\lambda x} \left[ C_1 \cos(\beta x) + C_2 \sin(\beta x) \right] - n_1 \frac{dF_t(x)}{dx} - n_2 q \tag{4}
\]

The constants in Eqs. (3) and (4) are as follows:

\[
B_1 = \frac{k_T y_b q a}{E_s I_b 2 \lambda} (L-a) - \frac{k_f}{\lambda^2} \left( \frac{d_s}{E_s I_b + E_f I_f} \right) q \tag{5}
\]

\[
B_2 = -B_1 = \frac{k_T y_b q a}{E_s I_b 2 \lambda} (L-a) + \frac{k_f}{\lambda^2} \left( \frac{d_s}{E_s I_b + E_f I_f} \right) q \tag{6}
\]

\[
C_1 = \frac{k_T}{2 \beta^2 E_s I_b} \left[ V_r(0) + \beta M_r(0) \right] - \frac{k_f}{2 \beta^2} \left( \frac{y_b}{E_s I_b} - \frac{y_f}{E_f I_f} \right) F_t(0) + \frac{k_T q \lambda^2}{2 \beta^2} \left( \frac{y_b E_f I_f - y_f E_s I_s}{E_s I_b + E_f I_f} \right) \left( \frac{y_b a}{E_s I_b} (L-a) - \frac{1}{\lambda^2} \left( \frac{d_s}{E_s I_b + E_f I_f} \right) \right) \cos(\lambda x) \tag{7}
\]

\[
C_2 = -\frac{k_n}{2 \beta^2 E_s I_b} \left[ M_r(0) \right] + \frac{k_T q \lambda^2}{2 \beta^2} \left( \frac{y_b E_f I_f - y_f E_s I_s}{E_s I_b + E_f I_f} \right) \left( \frac{y_b a}{E_s I_b} (L-a) - \frac{1}{\lambda^2} \left( \frac{d_s}{E_s I_b + E_f I_f} \right) \right) \cos(\lambda x) \tag{8}
\]
\[ \lambda^2 = k\left( \frac{d^2}{E_I b^2 + E_h A_f} + \frac{1}{E_h A_h} + \frac{1}{E_f A_f} \right) \]  
(9) 
\[ m = \frac{k}{\lambda^2} \left( \frac{d_o}{E_I I_b + E_h I_f} \right) \]  
(10) 
\[ \beta = \frac{k}{4} \left( \frac{1}{E_I b} + \frac{1}{E_f I_f} \right) \]  
(11) 
\[ n_1 = \frac{y_h E_I I_f}{E_o I_b + E_f I_f} \]  
(12) 
\[ n_2 = \left( \frac{E_f I_f}{E_o I_b + E_f I_f} \right) \]  
(13) 

where \( M_r(0) = \frac{qa}{2}(L-a) \) and \( V_r(0) = q\left( \frac{L}{2} - a \right) \) are the total bending moment and the total shear force in the strengthened beam at \( x=0 \) (i.e. at the NSM bar end) respectively, \( L \) and \( a \) are the span of the beam and the distance from the bar end to the nearest support respectively.

As can be seen from Eqs. 3 to 13, most parameters are directly associated with either the material properties of FRP/concrete or the geometric properties and can thus be easily obtained, except for (1) the two interfacial stiffness parameters (i.e., \( k_t \) and \( k_n \)), and (2) the eccentricity of tangential interaction force (i.e., \( y_f \)) to the centroid of the NSM bar. Zhang and Teng (2013) proposed equations of the above-mentioned parameters for RC beams strengthened with NSM rectangular bars and this paper provides the equations for RC beams strengthened with NSM round bars.

**INTERFACIAL STIFFNESS PARAMETERS**

In the present study, the interfacial stiffness parameters were obtained directly based on their definitions by making use of FE models developed in ABAQUS (2012). According to the definitions of interfacial stiffness parameters, the interaction force between the NSM bar and concrete per unit length needs to be found out for a unit relative displacement between the two in the designated direction. Therefore, the FE models consisted of an NSM round bar, the surrounding adhesive layer and the groove surface which was set to be a fixed boundary of the adhesive layer. With such FE models, the interfacial stiffness was just equal to the total reaction force of the boundary (i.e. the groove surface) when a unit displacement (i.e. 1 mm) was applied to the NSM bar in the designated direction. The schematic of the FE models used to obtain the interfacial stiffness parameters is shown in Figure 2.

(a) Unit-length segment          (b) Tangential interfacial stiffness \( k_t \)          (c) Normal interfacial stiffness \( k_n \)

![Diagram](image)

Figure 2. Schematic of the FE models used for the determination of interfacial stiffness parameters

In determining the interfacial tangential stiffness (Figure 2b), 8-node solid elements with a full Gauss integration were used and the adhesive layer is restrained against displacements in the two transverse detections (i.e. only displacements in the longitudinal direction were allowed). To save the computation time, the length of the models was set to be 0.1 mm instead of the unit length of 1 mm; the reaction force obtained from these FE models was then multiplied by 10 to obtain the reaction force corresponding to a unit length. As the adhesive layer was under pure shear, such treatment does not affect the accuracy of the results. In evaluating the normal interfacial stiffness (Figure 2c), the problem was simplified as a plain strain problem in the transverse direction and 4-node quadrangle
plane strain elements with a full Gauss integration were used, considering that the length of the FRP bar is usually much larger than the cross-sectional dimensions of the groove. Regression analyses of the FE results obtained from the numerical parametric studies using the above-mentioned FE models were conducted and the best-fit equations for the interfacial stiffness and eccentricity are as follows:

\[ k_t = AG_a = 7.1G_a(R - 1)^{0.61} = 7.1G_a \left(\frac{1}{R - 1}\right)^{0.61} \]  
\[ (14) \]

\[ k_n = BE_a = 4E_a(R - 1)^{0.66} = 4E_a \left(\frac{1}{R - 1}\right)^{0.66} \]  
\[ (15) \]

\[ y_f = \frac{0.165}{W^{0.72}D^{0.72}} \]  
\[ (16) \]

Where \( D \) is the diameter of the FRP bar, \( W \) is the side length of the square groove, and \( R \) is the ratio of \( W/D \).

VERIFICATION OF THE ANALYTICAL SOLUTIONS

To verify the proposed analytical solutions (Eqs. 3 to 13), 3D FE analysis of a RC beam strengthened with an NSM round bar under a UDL were conducted using ABAQUS (2012) and the results are compared with analytical predictions. The modelled RC beam had a span of 2400mm, a height of 300mm and a width of 150mm. The diameter of the NSM round bar was 8 mm and the groove side length was 12 mm, leading to a length-to-diameter ratio of 1.5. The NSM round bar was symmetrically placed with respect to the mid-span of the beam, with a length of 1800mm; the distance from the NSM bar end to the nearest support was thus 300mm. A UDL of 30 N/mm was applied on the top surface of the beam. By taking advantage of symmetry, only a quarter of the beam was modelled, with symmetric boundary conditions applied (Figure 3). All of the materials were modelled using 8-node solid elements with a full Gauss integration.

The interaction forces were obtained by integrating the relevant stresses along a prescribed path in the adhesive layer. The interaction forces obtained from the FE model are compared with the predictions from Eqs. 3 to 13 in Figure 4, from which it can be seen that the analytical predictions of both tangential interaction forces and normal interaction forces agree very well with the FE predictions, with gaps only existing in a very small region close to the NSM bar end. The errors of the analytical solution in that small region is due to the fact that the analytical solution does not take into account the boundary condition of zero shear stress at the bar end.
CONCLUSIONS

An analytical solution to interaction forces between the NSM bar and concrete in RC beams strengthened with NSM round bars are presented in this paper, following the work conducted by Zhang and Teng (2013) for RC beams strengthened with NSM rectangular bars. The key elements of the proposed analytical solution are the two interfacial stiffness parameters (i.e. tangential interfacial stiffness and normal interfacial stiffness) and the eccentricity of the tangential interaction force. FE models developed using ABAQUS (2012) were adopted to conduct numerical parametric studies, the results from which were used to determine the interfacial stiffness parameters and eccentricity of the tangential interaction force. The accuracy of the proposed analytical solutions has been verified with a sophisticated 3D FE model of a RC beam strengthened with a NSM round bar. Both analytical predictions and FE predictions identified high interaction forces near the NSM bar end, which to some extent explain why end debonding failures commonly happen in RC beams strengthened with NSM round bars.

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ABSTRACT

Near surface mounted (NSM) fiber-reinforced polymer (FRP) technique has been proved to be a reliable rehabilitation and strengthening method for deficient concrete structures. In such strengthened structures, debonding between FRP and concrete substrate is a typical failure model. This paper presents the simulation results for the bond behavior under direct pullout condition throughout the debonding process using a comprehensive three-dimensional finite element model. The concrete is simulated using concrete damage plasticity model while the FRP and adhesive are treated as linear-elastic materials. Tie joints are used at the adhesive-FRP and adhesive-concrete interface. Debonding is assumed to be caused by the concrete damage occurring in the region near the interface of adhesive-concrete. Instead of common static analysis approach, dynamic approach is introduced to overcome convergence problem, which provides more stable and accurate results than the static approach. Numerical results obtained are in close agreement with the available test data. The modeling approach proposed in this paper is not only applicable to NSM FRP system, but also offers a reliable tool to investigate characteristics of other systems relate to FRP and concrete bond.

KEYWORDS

Bond, fiber-reinforced polymer (FRP), finite element, near-surface mounted (NSM).

INTRODUCTION

Near surface mounted (NSM) fiber-reinforced polymer (FRP) is a reliable rehabilitation and strengthening technique for deficient concrete structures (e.g. De Lorenzis and Nanni 2002). Such strengthened concrete structures often fail by debonding between FRP and concrete substrate. Numerical simulation that can capture such a debonding mechanism is essential for predicting the capacity and investigating the bond characteristics of NSM FRP in concrete. Extensive studies for debonding simulation have been conducted on the externally bonded (EB) technique. These simulations typically employ two-dimensional finite element (FE) models (e.g. Chen et al. 2011; Kotynia et al. 2008; Lu et al. 2005) because the stresses in the EB application occur approximately within the plane along the direction of the bonding interface and its vertical component. However, for NSM technique, the strengthened member experiences the hoop and radial stresses in addition to the stress along the bonding interface. The stresses of NSM FRP occur in the three-dimensional space, which are more complex than EB FRP. As a result, these two-dimensional modelling approaches for the EB FRP cannot be employed to simulate the NSM FRP debonding without modification. Several existing studies use three-dimensional FE models (Cruz et al. 2006; De Lorenzis et al. 2004; Echeverria and Perera 2013; Hawileh 2012; Sasmal et al. 2013). Interfacial element and spring element are deployed between the FRP and concrete substrate for modelling the adhesive layer. Both of these elements are specified by using a constitutive law of bond-slip relationship from physical debonding tests. However, the results show that, by using this approach, the FE models could only simulate the debonding process but were unable to predict the capacity and investigate the bond characteristics. Teng et al. (2013) employed a more practical FE model in which concrete, FRP, and adhesive layer are modelled with solid elements. Teng’s model described a more realistic mechanical state of debonding than the other existing models, and is not limited to simple debonding simulation only. However, certain deficiencies such as failure model of adhesive and size reduction could reduce its accuracy and restrict the general applicability of the model itself.

This paper proposes an improved finite element modelling approach for the bond behaviour of NSM FRP under direct pull-out condition throughout the debonding process. Simulation results, which compare well with the available test data, are presented to prove the capability and accuracy of the proposed approach.
PROPOSED FE MODEL FOR NSM FRP-CONCRETE BOND JOINT

General

The FE model in this study is constructed using FE software ABAQUS (Dassault Systems Corp. 2014). Only half of the specimen along the width is modelled due to the symmetric configuration of the typical direct pull-out test, where appropriate boundary conditions are applied on the plan of symmetry. Size simplification, which was used in several other studies (e.g., Echeverria and Perera 2013; Teng et al. 2013), is not considered herein because it was reported to lead to inaccurate stress state (Yao et al. 2005).

The first-order hexahedral stress element in ABAQUS is used to model the FRP, concrete and adhesive layer materials. These three parts are connected to each other through the tie contact function in ABAQUS. Although debonding could be caused by either adhesive failure or concrete failure near the adhesive–concrete interface, the latter is found to be more common in physical testing and therefore is considered in the current modeling. The FRP and adhesive materials are assumed to behave linear-elastically and the concrete is modeled using the concrete damage plasticity model in ABAQUS to simulate its nonlinear behavior.

Modelling of Concrete

Constitutive law

For concrete under uniaxial compression, according to CEB-FIP (1990), the stress-strain relation shown in Figure 1 is composed of two branches described by the following set of equations:

\[
\sigma = \begin{cases} 
\frac{E_{ci} \varepsilon_c}{E_{ci}} - \left(\frac{\varepsilon_c}{\varepsilon_{c1}}\right)^2 & \text{for } |\varepsilon_c| < |\varepsilon_{c,lim}| \\
1 + \frac{E_{ci} - 2}{E_{ci}} \frac{\varepsilon_c}{\varepsilon_{c1}} - \frac{f_{cm} \varepsilon_c}{E_{ci}} & \text{for } |\varepsilon_c| > |\varepsilon_{c,lim}| 
\end{cases}
\]

(1a)

\[
\sigma_c = -\left(\frac{1}{\varepsilon_{c,lim}/\varepsilon_{c1}} - \frac{2}{\varepsilon_{c,lim}/\varepsilon_{c1}^2}\left(\frac{\varepsilon_c}{\varepsilon_{c1}}\right)^2 + \frac{4}{\varepsilon_{c,lim}/\varepsilon_{c1}} - \xi\right)\left(\frac{\varepsilon_c}{\varepsilon_{c1}}\right)^{-1} f_{cm} \varepsilon_c \quad \text{for } |\varepsilon_c| > |\varepsilon_{c,lim}|
\]

(1b)

where \(\sigma_c\) is the compression stress, \(\varepsilon_c\) is the compression strain, \(f_{cm}\) is the compressive strength, \(\varepsilon_{c1}\) is the correspond strain of \(f_{cm}\) (typically given as 0.0022), \(E_{ci}\) is the tangent modulus, and \(E_{c1}\) is the secant modulus from the origin to \(f_{cm}\).

\[\xi = \frac{4 \left(\frac{\varepsilon_{c,lim}}{\varepsilon_{c1}}\right)^2 \left(\frac{E_{ci}}{E_{c1}} - 2\right) + 2 \frac{\varepsilon_{c,lim}}{\varepsilon_{c1}} - \frac{E_{ci}}{E_{c1}}}{\left[\frac{\varepsilon_{c,lim}}{\varepsilon_{c1}} \left(\frac{E_{ci}}{E_{c1}} - 2\right) + 1\right]^2} \quad \text{(2a)}\]
In the equations above, $E_{ci}$ is typically estimated from the compressive strength of concrete $f_{cm}$, i.e., $E_{ci}=E_{00}[f_{cm}/f_{cm}]^{1/3}$, where $E_{c0}$ and $f_{cm0}$ are constants set to be equal to 2.15x10^4 MPa and 10 MPa per CEB-FIP (1990), respectively.

For the tension part of the concrete behavior under uniaxial tension, the ascending branch in the constitutive relation is assumed to be linear-elastic up to the stress equaling concrete’s tensile strength, and the softening branch employs the curve proposed by Nordijk (1991) as follows:

$$\frac{\sigma_c}{f_t} = \left[1 + \left(\frac{3w_c}{w_f}\right)^{1/3}\right] \exp\left(-6.93\frac{w_c}{w_f}\right) - 28\frac{w_c}{w_f}\exp(-6.93)$$

$$w_c = 5.14G_f/f_t$$

where $\sigma_c$ is the tensile stress, $f_t$ is the tensile strength, $w$ is the crack opening width, $w_c$ is the ultimate crack opening width, and $G_f$ is the fracture energy. $f_t$ and $G_f$ can be estimated based on CEB-FIP (1990) if no test data is available, i.e.,

$$f_t = 1.4\left(\frac{f_{cm0} - 8}{10}\right)^{2/3}$$

$$G_f = \left(46.875 \times 10^{-6} d_{max}^2 - 0.0005 d_{max} + 0.026\right)(f_{cm0}/10)^{0.7}$$

where $d_{max}$ is the maximum aggregate size. In the present model, $d_{max}$ is assumed to equal 20 mm.

**Damage model**

The concrete damage plasticity model in ABAQUS is adopted to simulate the nonlinear behaviour of concrete after cracking to avoid the phenomenon of “strain localization” due to the well-known drawback in the smear crack model in ABAQUS. The ratio of $r_t$ to $r_c$, defined by $K$, is assumed to equal 0.66, in which $r_t$ is the distance between the tension meridian line and the hydrostatic pressure axis, and $r_c$ is the distance between the compression meridian line and the hydrostatic pressure axis. The ratio of the initial yield stress under equibiaxial loading to the initial yield stress under uniaxial loading, $f_{s0}/f_{cm0}$, is set to be equal to 1.16 (Lubliner et al. 1989). These two ratios are used to define the yield field $F(\sigma, \dot{\varepsilon}^s)$ in the damage plasticity model. The other two coefficients, the dilation angle, $\Psi$, and the eccentricity, $\epsilon$, are assumed to be 30 and 0.1, respectively. These two coefficients are needed to define the flow potential $G$ in the damage plasticity model.

In addition to the yield field and the flow potential, the compressive damage variable, $d_c$, and the tensile damage variable, $d_t$, need to be defined although the model is only subject to monotonic loading (An 2015). The damage variables are estimated by the following:

$$d_c = 1 - \frac{\sigma_c/E_{ci}}{\varepsilon_c^u(1-b_c) + \sigma_c/E_{ci}}$$

$$d_t = 1 - \frac{\sigma_t/E_{ti}}{\varepsilon_t^u(1-b_t) + \sigma_t/E_{ti}}$$

where $\varepsilon_c^u = \varepsilon_c - \sigma_c/E_{ci}$ is the inelastic strain for concrete under uniaxial compression, and $\varepsilon_t^u = \varepsilon_t - \sigma_t/E_{ti}$ is the cracking strain for concrete under uniaxial tension. $b_c$ and $b_t$ are two variables related to strain, which are assumed to be constants equalling 0.7 and 0.1, respectively (Birtel and Mark 2006). Note that Eq. (7b) specifies $d_t$ using the stress-strain instead of the stress-displacement relationship. As a result, the crack bond model proposed by Bazant and Oh (1983) is applied to transform the stress-displacement curve defined by Eq. (3) into a stress-strain curve. The crack bond model assumes that the tensile fracture of concrete occurs in the form of a blunt smeared crack band in which the concrete undergoes progressive microcracking characterized by a stress-strain relation that exhibits strain softening. Hence, the relation of the crack opening width, $w$, the crack band width, $h_c$, and the crack strain, $\varepsilon_c^s$, can be expressed using the following equation based on the crack band model:

$$w = \int_0 h_c \varepsilon_c^s dh$$

The crack bandwidth $h_c$ in Eq. (8) can be estimated using $\sqrt{2e}$, in which $e$ is the element length (Rots and Blauuwendraad 1989).
Modelling of FRP and Adhesive Layer

As previously mentioned, debonding is assumed to occur in the concrete adjacent to the adhesive-concrete interface. Therefore, the FRP and adhesive are treated as linear-elastic material. Their material properties, such as tensile strength and Poisson’s ratio, are obtained from the material coupon tests in the lab (to be discussed in the next section).

Solution Method

For conventional static solution techniques, there is a challenge that lies in the convergence issue of the FE model during later stages of loading. These techniques such as the Newton-Raphson method are insufficient to handle the strain softening phenomena described by the material constitutive law. Solution process is often interrupted when concrete starts to experience damage. Some experimental observations, such as intermittent crack noises during debonding process and quick energy release at debonding failure, have suggested that the debonding failure should be treated as a dynamic problem instead of a static problem. Chen et al. (2015) proved this hypothesis and proposed a dynamic solution approach to overcome the convergence challenge when simulating the strain softening behaviour of concrete. In their study, both the implicit HHT-α method and the explicit central difference method were investigated based on a FE modelling for a FRP-strengthened reinforcement concrete (RC) beam. The former method was found to provide a better performance than the latter on accuracy and stability. However, the implicit HHT-α method does not exhibit its potential in the present study as claimed in their study (Chen et al. 2015). It even failed due to a severe convergence issue similar to the conventional static solution technique. Therefore, the central difference method is employed herein as a viable solution technique.

For a dynamic approach, two key elements exist, the damping scheme and the loading time. The Rayleigh damping is adopted in the present model to define the damping scheme, where the damping matrix $C$ can be estimated by the following:

$$C = \alpha_0 M + \alpha_1 K$$

(9)

where $M$ and $K$ are, respectively, the mass matrix and the stiffness matrix. $\alpha_0$ and $\alpha_1$ are the proportionality constants for mass component of the damping and stiffness component of the damping, respectively. These two constants can be determined by the damping ratios of any two modes as follows:

$$\alpha_0 = \frac{2\omega_i\omega_j (\zeta_i\omega_i - \zeta_j\omega_j)}{\omega_i^2 - \omega_j^2}$$

(10a)

$$\alpha_1 = \frac{2(\zeta_j\omega_j - \zeta_i\omega_i)}{\omega_j^2 - \omega_i^2}$$

(10b)

where $\omega$ and $\zeta$ are the damping ratio and the circular frequency, respectively. Their subscripts represent the order of the mode. In the present study, the first and second modes are used to determinate $\alpha_0$ and $\alpha_1$ by assuming the same damping ratio for all modes. Although the values of the damping ratio, $\zeta$, and the loading time, $t$, are typically obtained based on a trial-and-error process, it is known that the loading time, $t$, should increase proportionally with the damping ratio, $\zeta$, in order to obtain an accurate solution (Chen et al. 2015). Therefore, a small value of $\zeta$ is recommended in order to reduce the computational effort.

Numerical Results

The direct pull-out debonding behavior reported by Li et al. (2005) is simulated using the proposed FE approach in this study for verification purpose. In this direct pull-out experiment, five NSM FRP strip-concrete bonded joints were tested. The concrete component consisted of a rectangular block with a square section of 150x150 mm and a length of 350 mm, as shown in Figure 2(a). The groove size had a width of 9 mm and a length of 22 mm. The bond lengths involved were 30 mm, 100 mm, 150 mm, 200 mm and 500 mm. The groove-filling material was a two-component epoxy adhesive. More details are provided in Table 1 (Li et al. 2005). The major results for specimen CS-150 with a bond length of 150 mm are selected herein for detailed discussion and the complete set of results is reported in separate publications.

Figure 2(b) below compares the load-displacement response at the loaded end of the specimen. Note that Li et al. (2005) only reported the ultimate pull-out load from the test (no load-displacement curve was available). It can be seen that the predicted ultimate pull-out load is in close agreement with the test results.
Table 1 Test details (Li et al. 2005)

<table>
<thead>
<tr>
<th>CFRP strip</th>
<th>Concrete</th>
<th>Adhesive</th>
<th>Ultimate load</th>
</tr>
</thead>
<tbody>
<tr>
<td>thickness (mm)</td>
<td>width (mm)</td>
<td>modulus (GPa)</td>
<td>Poisson’s ratio</td>
</tr>
<tr>
<td>5</td>
<td>16</td>
<td>120.8</td>
<td>0.2</td>
</tr>
</tbody>
</table>

Figure 2 Details of: (a) geometry and boundary conditions; and (b) comparison of load-displacement response

Figure 3(a) displays the distribution of the local bond stress for various loading cases including 20%\(P_u\), 45%\(P_u\), 70%\(P_u\), 95%\(P_u\), and 100%\(P_u\) in the loading branch (where \(P_u\) is the ultimate load). The distribution of the corresponding tensile strain in the FRP strip in the loading direction is shown in Figure 3(b). Typical distributions of the local bond stress and FRP tensile strain can be seen in this figure. The local bond stress in Figure 3(a) exhibits a parabolic-shaped distribution in the loading direction, and its peak value progressively increases and moves away from the free end (i.e., towards the loaded end) as the load increases. Correspondingly, the tensile strain in the FRP strip increases nonlinearly towards the loaded end, but its gradient decreases in the region near the loaded end as the load increases. Although the predicted bond stress distribution is slightly different from the distribution measured in the test reported by Li et al. (2005), the predicted peak values for all the cases are in close agreement with the test data. The predicted tensile strain distribution represents a similar pattern to the test result (comparison is not included herein).

Figure 3 Distribution of: (a) local bond stress; and (b) FRP tensile strain

The predicted debonding failure pattern is illustrated in Figure 4, which shows a distinct spine-shaped concrete damage contour. This spine-shaped concrete damage contour is not only found in Li et al.’s tests (2005), but also observed in other existing pull-out experiments (e.g., Lee et al. 2013). Figures 5(a) and 5(b) illustrates the observed failure modes of two representative specimens by Li et al. (2005) and Lee et al. (2013), respectively.
CONCLUSIONS

This paper proposes an improved three-dimensional FE modelling approach for bond behaviour of NSM FRP under direct pull-out condition throughout the debonding process. Concrete damage is simulated using the concrete damage plasticity model while the FRP and adhesive are treated as linear-elastic materials. An explicit dynamic solution method is introduced to overcome the convergence issue often encountered in static method and implicit dynamic method. Numerical results showed that the proposed FE modelling approach could simulate the debonding failure of NSM FRP to a good accuracy.

REFERENCES

THE TORSIONAL STRENGTHENING OF RC BEAMS USING THE NSM FRP TECHNIQUE WITH FRP ROPE

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ABSTRACT

The near surface mounted (NSM) strengthening technique possesses many advantages over externally bonded (EB) when it comes to strengthening of reinforced concrete (RC) beams using fiber reinforced polymer (FRP) composites. However, the majority of the research on NSM FRP has been confined to flexural and shear strengthening applications and no considerations for torsional strengthening have been made. The present study aims to present an experimental program that focuses on the torsional strengthening of RC beams using the NSM technique. Alternative torsional strengthening configurations were tested with full wrapping using FRP rope. Four RC beams have been tested where the results from two RC beams are presented and compared with unstrengthened control specimens. The beams were tested using a specialised torsional actuator and the results expressed in terms of the maximum torsional capacity with respect to the peak angle of twist. The results demonstrated that the NSM technique with FRP rope was an effective method to increase the torsional strength of RC members.

KEYWORDS

Torsion, RFP rope, grooves, NSM, concrete beam, epoxy, strengthening.

INTRODUCTION

In order of the reinforcement concrete beams to be generally strengthened, sheets of carbon fiber reinforced polymers (CFRP) are applied on the faces of the concrete using an externally bonded reinforcing (EBR) technique to increase the torsional resistance of beams (Fib Bulletin 14 2001, Hii 2006). The increase of the torsional resistance of concrete beams by adopting the EBR technique has been verified by many researchers (Hii and Al-Mahaidi 2006; Mohammadizadeh et al. 2009; Deifalla and Ghobarah 2010). On other hand, the maximum strain measured by using this technique is below the ultimate strain due to premature debonding. In an attempt to solve this problem, near surface mounted technique has been proposed where CFRP laminates are installed into thin pre-cut grooves (Nanni et al. 2004). This strengthening technique has already proven to be effective for the shear strengthening of reinforced concrete beams (Barros and Dias 2006; El-Hacha and Wagner 2009; Lim 2010). In the experimental program carried out by Barros and Dias (2006), the efficiency of the NSM technique by testing four series of beams has been investigated. Each group comprised five beams. One beam was strengthened with CFRP sheet strips applied according to the EBR technique to compare the efficacy of the EBR and NSM techniques. The researchers demonstrated that the NSM shear strengthening technique was the easiest to apply and the most effective of the CFRP techniques for both beam load-carrying capacity and deformation capacity. In another experimental program, seven reinforced concrete beams with a rectangular cross-section were cast and tested by El-Hacha and Wagner (2009) to investigate the effectiveness of vertical NSM CFRP strips. The NSM CFRP strips were applied for shear strengthening at different spacings on RC beams weak in shear. It was reasoned that the effective depth of the beams should be greater than the spacing between the strips to avoid brittle failure in shear before yielding of the longitudinal steel. Then, Lim (2010) examined the shear strengthening effectiveness of nine beams strengthened with NSM CFRP strips and EBR CFRP strips. One beam was not strengthened and tested as a control, and five beams were tested as beams strengthened with EBR FRP strips with different spacings. Three beams were strengthened using NSM FRP strips. One was strengthened with 25 mm NSM with a spacing of 150 mm, whereas the rest were strengthened with the combined use of NSM and EBR CFRP strips. It was concluded that shear stiffness and strength were considerably developed for the beams strengthened by combined NSM and EBR compared with the control beam. The first and the only study conducted by Al-Bayati et al. 2016 to evaluate the effectiveness of using NSM technique with torsion according to the authors’ knowledge. Ten beams have been tested, two control beams and eight beams strengthened with CFRP laminates applied to all four faces of the beams. Epoxy adhesive was used in four strengthened beams and modified cement-based adhesive was used as a substitute
for epoxy in the remaining four beams. Two different spacings between the grooves (0.75D and 0.375D where D is the depth of the beam) were examined for each type of adhesive. According to their results, the torsional behaviour of the beams was improved in the strengthened beams by almost 28.22% and 35.93% when using the epoxy for 0.75D and 0.375D spacings of CFRP NSM laminates respectively. On the other hand, the NSM technique with the cement-based adhesive was also an effective method in increasing the torsional strength by by 23.4% and 26.5% for 0.75D and 0.375D spacings respectively.

In the present paper, a similar technique of the torsion strengthening was used with the difference that, instead of CFRP laminates at four faces, FRP rope around the cross section of the beam with the equivalent carbon fiber percentage was used. Four beams have been prepared and tested comprising of two matching control beams and an additional two beams strengthened in torsion using FRP rope and epoxy. Similar strengthening technique was used for the two strengthened beams in order to confirm the consistency of the results.

**EXPERIMENTAL PROGRAM**

**Beam prototype and strengthening configuration**

Four rectangular reinforced concrete beams with 260 mm by 140 mm in cross section and 2000 mm long were poured; two comparable control beams and two were identically strengthened using the NSM technique with epoxy resin. The design of the beams was performed in order to fail in torsion in the middle 1.2 m section of each beam which allowed more than one torsional spiral crack to form at 45° angles with respect to the beam longitudinal axis. Figure 1 presents the dimensions of the beams’ cross section and the steel reinforcement details. The beams were under-reinforced according to the Australian design code AS3600-2009 (SAI 2009) to simulate beams that were torsionally deficient under some future loading condition. In addition, it was decided to intentionally exceed the minimum spacing of the hoop reinforcement specified by the design code in order to minimize the stirrups ability to restrict the torsional cracks, thus facilitating the observation of torsional failure. FRP ropes were embedded into grooves cut around the beam cross section within the 20 mm cover zone of the concrete beams. The groove dimensions are 5 mm wide and 18 mm deep respectively. In order to make a comparison with the results of the beam strengthened using externally FRP strips (Hii and Al-Mahaidi 2004), it was decided that the spacing between the grooves will be 195mm (0.75D where D is the beam depth) as shown in Figure 2, which is the same strips’ spacings used by Hii and Al-Mahaidi 2004. In addition, the cross section of the FRP rope used in each groove was selected to provide the same FRP area of the 50 mm width FRP wraps used by Hii and Al-Mahaidi 2004.

Material properties and beams preparation

The compressive strength of the supplied concrete by a local supplier was evaluated at 28 days and at the age of the beam tests, executing direct compression tests with cylinders of 100 mm diameter and 200 mm height, according to ASTM C873/C873M-15. In the tested beams, steel reinforcement bars of 6 mm diameter were used. Three bars of each diameter were selected and uniaxial tensile tests were carried out to obtain the Elastic modulus (Es), yield and ultimate strength (fy, fu) according to the recommendations of ASTM A370-10 2010. The mechanical properties of CFRP rope were characterised through uniaxial tensile tests using a 250 kN MTS machine carried out according to the ASTM D3039/D3039M (2008) and BS EN ISO 527-5 (2009) recommendations. Strain gauges were used to obtain the CFRP strains during the test. The Sikadur-330 epoxy was used as adhesive to fill the grooves, while The Sikadur-300 epoxy were used as impregnating resin. The Sikadur-330 is a 2 parts light grey epoxy, the Sikadur-300 is however a 2 parts epoxy clear and liquid when mixed.

Table 1 includes the mechanical properties of all materials, which summarises the average compressive strength, tensile strength and modulus of elasticity of the concrete. The tensile strength of the epoxy was also tested and the modulus of elasticity was based only on specifications supplied by the manufacturer. Three reinforcement bars for each diameter and the five samples of CFRP rope were tested and the average values measured from these experimental programs were presented in this table.
In all beams, electrical resistance strain gauges (length 5 mm) were used to monitor the strain and verify whether yielding in the selected steel stirrups had occurred. Four strain gauges were used for each beam and placed on north legs of the stirrups (tension legs), which were titled S1-S4 as shown in Figure 3 (a).

To apply the pre-cured CFRP rope using the NSM technique After 28 curing days of the beams, the following procedures were carried out: (1) grooves were opened on the concrete cover of the all faces of the beam using a diamond chaser; (2) the grooves were cleaned by brush and vacuum cleaner to remove dust and any loose materials; (3) the ropes were measured and cut with the desirable length to surround the cross section of the beam with enough overlap; (4) The Sikadur-330 and Sikadur-300 epoxy was produced according to the supplier recommendations; (5) the FRP rope was impregnated with Sikadur-300 epoxy resin until completely saturated then the air and the excess resin was squeezed out of the rope; (6) The groove around the cross section was filled with Sikadur-330 adhesive using a plastic blade; (7) The FRP rope was carefully inserted into the groove around the cross section to the bottom, the lateral faces and the top of the beam consecutively with the use of the plastic blade until reaching the bottom of the groove and the plastic blade was carefully removed without pulling the rope back out; (8) The surface was levelled and excess adhesive that has been squeezed out of the groove was removed. To guarantee a proper curing of the epoxy, already enough time (90 days) passed between the beam strengthening operations and the test. Figure 3 (b) presents photos for the preparation of the strengthened beams.

![Figure 3 The strain gauges’ locations and the preparation of the strengthened beams](image)

### Table 1 Mechanical properties of materials

<table>
<thead>
<tr>
<th>Material</th>
<th>Compressive strength (mpa)</th>
<th>Tensile strength (MPa)</th>
<th>Modulus of elasticity (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>54.28</td>
<td>3.83</td>
<td>40040</td>
</tr>
<tr>
<td>Sikadur-300</td>
<td>-</td>
<td>70.4</td>
<td>3450 flexural, 1724 tensile</td>
</tr>
<tr>
<td>Sikadur-330</td>
<td>-</td>
<td>31.9</td>
<td>3800 flexural, 4500 tensile</td>
</tr>
<tr>
<td>Steel bars ø12</td>
<td>-</td>
<td>FY: 544.5</td>
<td>FU: 692.4</td>
</tr>
<tr>
<td>Steel bars ø6</td>
<td>-</td>
<td>FY: 505.7</td>
<td>Fu: 600.7</td>
</tr>
<tr>
<td>FRP rope</td>
<td>-</td>
<td>-</td>
<td>FU:4050</td>
</tr>
</tbody>
</table>

To apply the loading arm was used to apply torsion on the one end of the beam where another end was fixed to the support as illustrated in Figure 4. As the focus was just on the pure torque, the loading arm was ensured to be in line with the end spherical support to prevent flexure from being applied on the beam. At the fixed end, a steel frame was used consisting of two half I crossbeams bolted to two steel columns. Each column was fixed to the 1 m thick reinforced concrete strong floor using anchor bolts as shown in Figure 4 (a). Flanges and collar were used to anchor the beam to the fixed frame as no longitudinal, vertical, transverse and rotational movements were allowed. At the loading end, a roller seat on a linear bearing with a spherical ball was used to support the beam where the load was applied which also permitted axial movement. This allowed twisting the beam freely while allowing elongation or shortening as presented in Figure 4 (b). A single 250kN capacity hydraulic actuator fixed to the loading frame was adopted to apply the torque through the loading arm. The roller seat and the loading arm were aligned to produce pure torsion and ensure that no other bending or shear forces were introduced into the system. Another spherical ball was located over the cantilever arm and under the hydraulic actuator to ensure that the load remained vertical during the test. To facilitate effective use of lab instruments, the beams were tested with the shorter sides facing
up and down. The orientation of the cross-section was unimportant as a pure torsion was applied. To avoid stress concentrations from developing between the steel flange and the top flange of the beam, minor grinding was provided to smooth out any imperfections between the interfaces. A photo of the test setup is shown in Figure 4. (c). Instrumentations were used to record the torque, angle of twist, and cracks. A load cell was located under the hydraulic actuator to determine the applied torque. The exact amount of twist was measured using inclinometers were placed at both ends of the beam as in reality, there is always a small compliance at the fixed end.

RESULTS AND DISCUSSION

Figure 5 shows the relationship between the applied torque and the angle of twist for all four beams. All beams experience a linear elastic response until the crack torque was reached then it was followed by a large increase in the angle of twist with gradual increase in torque until failure. The strengthened specimens exhibited higher peak torques and more ductile behaviours after peak torques than the control specimens. For beams C2-1, E2-195R-1, and E2-195R-2, the first cracks appeared at the south faces which and propagated to the other three faces. However, the first crack developed at the north face of the beam C2-2 then was observed at the south face. The strengthening of beams developed stiffer pre and post-cracking responses than the control beams. Photos taken after testing for the beams’ depicting failure zones are presented in Figure 6. All beams failed in torsion as the applied torque was constant along the length of the beams between the gripping systems. The location of failure showed no specific trend, however the exact location of first crack initiation and failure could be related to factors such as: reinforcement spacing, non-homogeneity of concrete properties and other factors immanent to imperfections in beam fabrication. The failure modes of the all beams were by yielding of the steel closed ties followed by local concrete cover spalling and concrete crushing. A few large cracks have been observed for the control beams and the largest cracks were located in the vicinity of concrete spalling zone. For the strengthened beams, the FRP ropes seemed to be well bonded to the concrete. The concrete cover delamination was observed with separation of the concrete that includes the FRP rope. A lot of relatively small uniformly distributed cracks occurred in the space between grooves. The larger cracks developing between the first and the third grooves and between the second and the fourth grooves for beams E2-195R-1 and E2-195R-2 at the east side of the beam passed through the second and the third grooves respectively. More cracks were developed in the space between the grooves. The main results of the crack load, ultimate torsional loads and corresponding angle of twists are included in Table 2. The average loads and twists for each beam type are presented and the increase in torsional strength with respect to the control beam is presented in between the brackets. The average crack torque of the strengthened beams has an increase in torsional strength of 21.4% than the control beams while the increase percentage of ultimate torque was 23.3%. However, the increase percentages of ultimate torques of the beams using the EBR technique was 25.2% (Hi and Al-Mahaidi 2004).
Table 3 shows the strain values of stirrups measured at the peak load level where S1 was the nearest strain gauge to the load arm. The results of S2 strain gauge in beams E2-195R-1 was removed as they were damaged during beam manufacture. The yield strain is 0.002254 for the stirrups. In all beams, at least one strain gauge shown that the strain of the bar was exceeded the yield strain before reaching the peak torque. In Figure 7, torque versus stain curves are presented for all beams. After spiral cracks started to form in the beams, the reinforcement strain values increased rapidly. The stiffness of the beams was reduced with increasing crack opening as more force was transferred to the stirrups. From the strain gauges results, it was observed that the load carried by the stirrups of the strengthened beams was generally small compared with the reference beams. At control beams torque levels, the strain levels measured were generally lower than in beams C2-1 and C2-2.

![Figure 6 Failure zone for the beams](image)

![Figure 7 Strain versus angle curves for beams C2-1, C2-2, E2-195R-1 and E2-195R-2](image)

<table>
<thead>
<tr>
<th>Beams</th>
<th>Crack load (kN.m)</th>
<th>Average</th>
<th>Twist at crack load (°)</th>
<th>Average</th>
<th>Ultimate load (kN.m)</th>
<th>Average</th>
<th>Twist at ultimate load (°)</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>C2-1</td>
<td>5.119</td>
<td>5.279</td>
<td>0.142</td>
<td>0.1785</td>
<td>8.521</td>
<td>8.6145</td>
<td>3.453</td>
<td>4.011</td>
</tr>
<tr>
<td>C2-2</td>
<td>5.439</td>
<td></td>
<td>0.215</td>
<td></td>
<td>8.708</td>
<td></td>
<td>4.569</td>
<td></td>
</tr>
<tr>
<td>E2-195R-1</td>
<td>6.106</td>
<td>6.408</td>
<td>0.369</td>
<td>0.36</td>
<td>10.652</td>
<td>10.621</td>
<td>4.593</td>
<td>5.0085</td>
</tr>
<tr>
<td>E2-195R-2</td>
<td>6.710</td>
<td>(21.4)%</td>
<td>0.351</td>
<td></td>
<td>10.590</td>
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Table 3 The strain values at peak torque

<table>
<thead>
<tr>
<th>Strain gauge</th>
<th>Strain × 10^6</th>
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<tbody>
<tr>
<td>S1</td>
<td>3407</td>
</tr>
<tr>
<td>S2</td>
<td>1543</td>
</tr>
<tr>
<td>S3</td>
<td>3192</td>
</tr>
<tr>
<td>S4</td>
<td>779</td>
</tr>
</tbody>
</table>

Yield strain of stirrups = 2254 × 10^6
CONCLUSIONS

Based on the experimental work results, the use of Near Surface Mounted (NSM) technique with FRP rope is viable for torsional strengthening of reinforced concrete beams. Using an epoxy adhesive, the results for the strengthened beams were evaluated and compared with the reference beams. The crack and ultimate torques of the beams have been significantly improved by an average of 21.4% and 23.3% respectively. It was also observed that the strain at the same level of torque for the strengthened beams was generally small compared with the control beams. More ductile behavior was observed for the strengthened beams. At failure, the critical torsional cracks of the strengthened beams developed through the middle of the groove where the delamination of the concrete cover with the FRP rope. Further investigation is warranted to study other parameters for beams strengthened using the NSM technique to increase the FRP contribution in strengthening.

ACKNOWLEDGMENTS

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REFERENCES

ABSTRACT

This paper presents experimental and numerical investigations about the fire behaviour of reinforced concrete (RC) beams flexurally strengthened with carbon fibre reinforced polymer (CFRP) strips applied according to the near surface mounted (NSM) technique. The experimental programme included fire resistance tests on NSM-CFRP strengthened RC beams protected with calcium silicate boards with varying thickness along their length. The numerical study comprised the development of three-dimensional (3D) finite element (FE) models in order to simulate the fire resistance tests. The models considered the variation with temperature of the thermal and mechanical properties of the constituent materials, and the CFRP-adhesive-concrete interaction was simulated by means of bi-linear bond-slip laws previously (and independently) calibrated by the authors for different temperatures. The numerical results were compared with experimental data in terms of temperatures, midspan deflections and time to CFRP debonding. The models provided accurate predictions of the thermo-mechanical fire response of the strengthened beams. Together with the experiments, it was shown that it is possible to exploit the CFRP mechanical contribution through a cable mechanism, provided that a thicker insulation is applied along the CFRP anchorage zones. In addition, the results obtained also confirmed (and allowed quantifying) the much better fire resistance performance of NSM compared to EBR. Indeed, with the former technique, it has been shown that it is possible to attain relatively long fire endurances using relatively modest fire insulation thicknesses.

KEYWORDS

Reinforced concrete beams, CFRP strengthening, NSM, fire resistance, experimental tests, numerical models, uncoupled thermal-mechanical analysis.

INTRODUCTION

Carbon fibre reinforced polymer (CFRP) materials have been successfully used to strengthen reinforced concrete (RC) structures through externally bonding strips or sheets to the surface of the members with epoxy-based adhesives (EBR - externally bonded reinforcement technique). During the last decade, the alternative near surface mounted (NSM) technique, in which CFRP strips/bars are inserted and bonded into slits cut inside the concrete cover, has been gaining increasing interest mainly due to its higher efficacy and better anchoring capacity when compared to EBR (e.g. Barros et al. 2007). However, regardless of the strengthening technique (EBR or NSM), the behaviour of CFRP systems at high temperature is a matter of concern due to the polymeric nature of the CFRP matrix, as well as of the epoxy adhesives used to bond it to the concrete substrate. In fact, the glass transition temperature ($T_g$) of CFRP and conventional epoxy adhesives (between 45 and 120°C) can be attained in outdoor applications and very easily and quickly exceeded in the event of a fire. A recent state-of-the-art review (Firmo et al. 2015b) showed that although considerable research has been undertaken to better understand the response of FRP-strengthened RC elements at high temperatures or in fire, there is a very limited number of studies on the fire behaviour of NSM-strengthened RC members (e.g. Burke et al. 2013, Palmieri et al. 2013). The available research indicated that considerable fire endurances can be attained by applying fire protection systems over the strengthening system; however, it was not yet possible to correlate the temperatures in the materials (namely in the adhesive) with the loss of structural effectiveness of the FRP system.

This paper presents experimental and numerical investigations about the fire behaviour of RC beams flexurally strengthened with NSM-CFRP strips. The objective is two-fold: (i) to evaluate the effectiveness of different fire protection schemes with thicker insulation layers applied in the anchorage zones of the strengthening system, and (ii) to improve the understanding about the fire performance of NSM-strengthening systems, more specifically regarding the maximum/critical temperatures attained in the adhesive when the mechanical contribution of the
CFRP system is lost. To this end, fire resistance tests were performed on NSM-strengthened RC beams protected with calcium silicate (CS) boards with varying thickness along their length. The numerical study included the simulation of the thermomechanical response of the tested beams by means of 3D finite element models.

EXPERIMENTAL INVESTIGATIONS

Test programme and description of the specimens

Four fire resistance tests were conducted in an intermediate scale furnace on RC beams flexurally strengthened with NSM CFRP strips and thermally insulated with CS boards. The beams were 150 cm long, 10 cm wide and 12 cm deep (cf. Figure 1) and were cast with ready-mixed concrete (average compressive strength in cylinders of 37 MPa at 279 days – age of testing). The internal steel reinforcement, with 1.5 cm of cover, consisted of four \( \phi 6 \) mm longitudinal rebars and \( \phi 6 \) mm transverse stirrups distanced of 6 cm. The strengthening system for flexure (supplied by S&P Clever Reinforcement) comprised two CFRP strips (cross section of 10 mm \( \times \) 1.4 mm, tensile strength of 2850 MPa, Young’s modulus of 168 GPa) and an epoxy-based adhesive (tensile modulus and tensile strength of 10 GPa and 14 MPa, respectively), providing a flexural strength increase of 111% (experimentally determined through 3-point bending tests at ambient temperature). The glass transition temperatures of the CFRP strips (\( T_g = 88 \) º C) and of the epoxy adhesive (\( T_g = 47 \) º C) were determined from dynamic mechanical analyses (based on the onset of the storage modulus curve decay).

The fire insulation systems consisted of CS boards, with thicknesses ranging from 25 mm to 50 mm, which were mechanically fixed to the bottom surface of the beams. As depicted in Figure 1, the fire protection exhibited variable thickness along the length, being typically thicker in the anchorage zones of the CFRP strips (thickness \( T \) in Figure 1) and thinner at the central zone (thickness \( t \) in Figure 1). The nomenclature adopted for the beams is related to the geometry of the insulation system: the first number corresponds to the dimension \( T \) and the second one to the dimension \( t \) (cf. Figure 1, both in millimetres). The following beams (insulation schemes) were tested: 0-0 (unprotected); 25-0; 25-25; and 50-25. A more detailed description of the fire protection system is provided in Firmo and Correia (2015a).

Figure 1 Geometry of the NSM-strengthened RC beams and position of the thermocouples: a) longitudinal view; b) midspan cross section (AA’).

The fire resistance tests were performed on a vertical furnace powered by propane gas and the tested beams were placed over its opened top (figure 2a). Concerning the thermal boundary conditions, the bottom face of the beams was exposed to the ISO 834 standard fire, whereas their top face was submitted to ambient temperature; the lateral faces of the beam specimens were thermally insulated. Consequently, only the bottom face was directly exposed to heat - the test conditions actually simulated the thermal exposure typically found on one-way slabs. Regarding the structural loading, all specimens were simply supported and subjected to a sustained load (comprising concrete blocks and steel plates) applied in a four point bending configuration (figure 2). The load level applied during the tests (70% of the design load at ambient temperature - according to the Eurocode 2) corresponded to 37% of the failure load of the strengthened beams and 78% of the load capacity of an unstrengthened member (ratios defined based on bending tests at ambient temperature).

The instrumentation consisted of an electrical displacement transducer for measuring the midspan displacement at the top (cold part) of the beams (\( \delta \), in figure 2b) and a set of 15 thermocouples type K, whose nomenclature and location are shown in figure 1. The fire resistance tests started with the application of the structural loading; after a period of about 30 min, set in order to guarantee the stabilization of the deflections, the bottom surface of the beams started to be thermally exposed to the ISO 834 standard fire. The burners of the furnace were turned off a few minutes after the CFRP strengthening system lost its structural effectiveness (which could be easily identified from the measurements of the midspan deflections).
Summary of results

**Temperatures along the bonded interface when the CFRP system debonded**

Figure 3a shows the temperatures along the bonded interface (measured by thermocouples T8 to T15, cf. Figure 1) when the strengthening system lost its structural effectiveness (instants detected on the midspan displacement vs. time curves - cf. Figure 4a). As expected, the beam with CS boards applied only in the anchorage zones of the NSM strengthening system (beam 25-0) exhibited very high temperatures at the central zone (left uninsulated) and considerably lower temperatures in the CFRP anchorage zones. The temperature distributions obtained in the uninsulated beam (beam 0-0) and those protected by a 25 mm thick CS board along the entire length of the CFRP system (beam 25-25) were roughly uniform, presenting lower temperatures at the CFRP ends due to the vicinity of the furnace walls. Although the above-mentioned distributions were expected, Figure 3a shows that the temperatures in the anchorage zones presented relatively high scatter. This scatter is clearly noticed in Figure 3b, where the average temperature along the hotter anchorage zone at the debonding instant is plotted. This figure highlights the complexity of the behaviour of NSM-strengthened systems under exposure to fire, which is related to (i) the thermal gradient existing along the depth of the slits, and also to (ii) the confinement effect on the strengthening system caused by the thermal expansion of the surrounding concrete. Albeit, the temperature distributions revealed that the debonding of the strengthening systems occurred when the temperature in the anchorage zones was considerably higher than the adhesive $T_g$: ranging from 111 °C ($2.4 \times T_g$) to 236 °C ($5.0 \times T_g$). Additionally, Figure 3b also shows that in beam 25-0 the CFRP system lost its structural effectiveness for considerably lower temperatures in the anchorage zones (compared to the remaining beams). This result might be related to the very high temperatures recorded along their central zone (unprotected length), where the CFRP-concrete bond was severely damaged, therefore increasing the bond stresses in the CFRP anchorage zones and thus explaining the occurrence of CFRP debonding for lower temperatures.

Figure 3 Temperatures in the adhesive when the CFRP strengthening system debonded: a) along the bonded interface (E – experimental, N – numerical); b) average along the anchorage length (20 cm, defined according to ACI 440.2R for 20 °C).
Figure 4a plots the midspan displacement increase of the beams as a function of the time of fire exposure. As expected, this displacement increased with time (and temperature) due to the stiffness (and strength) decrease of the materials. The loss of structural effectiveness of the CFRP strengthening system is marked in this figure (with a small circle), and can be clearly identified by an increase on the slope of the curves; in most cases a progressive slope increase was observed. The insulation systems applied along the strengthening system (beams 25-25 and 50-25) delayed the thermal induced degradation on the materials during the entire test, explaining their lower midspan displacement increase rate (before and after the CFRP loss). Figure 4b summarizes the time of fire exposure until the loss of the CFRP system and confirms the high susceptibility of the strengthening system when exposed to heat without any protection (beam 0-0) - the CFRP debonded after only 18 minutes. This figure confirms that the application of thicker layers of insulation material allowed extending the structural effectiveness of the strengthening system by exploiting the CFRP mechanical contribution through a cable mechanism (already observed in previous studies, namely in EBR systems cf. Firmo et al. 2015b).

The fire behaviour of EBR-CFRP strengthened beams was assessed in a parallel study conducted by the authors (Firmo and Correia 2015b), in which similar materials, fire insulation schemes and test conditions were adopted. The worse fire performance of the EBR technique is illustrated in Figure 5a, where it is shown that for all insulation schemes, the fire resistance of the EBR-strengthened beams is considerably lower than that of the NSM counterparts.

Figure 4 a) Midspan displacement increase vs. time of fire exposure (E – experimental, N – numerical); b) fire resistance of the NSM-CFRP strengthening system.

**NUMERICAL INVESTIGATIONS**

**Description of the models**

The numerical investigations comprised the simulation of the fire resistance tests described above. The objective was two-fold: (i) to improve the understanding about the fire behaviour of NSM-CFRP-strengthened beams, namely confirming the cable behaviour of the CFRP system during the fire resistance tests, and (ii) to further validate the concept of fire protection systems comprising thicker insulation layers in the CFRP anchorage zones.

To achieve the aforementioned goals, three-dimensional FE models of the four NSM-CFRP-strengthened RC beams (specimens 0-0, 25-0, 25-25 and 50-25) were developed using the commercial package ABAQUS. In order to reduce the computational costs, only half of the beams’ length was simulated. Figure 5b exemplifies the FE mesh (with a maximum dimension of 10 mm) in which 8 node isoparametric hexahedral elements (DC3D8 and C3D8 for the thermal and mechanical analyses, respectively) were used to model concrete, CFRP strips and CS boards. For the steel rebars, 2-node truss elements (DC1D2 and T2D2 for the thermal and mechanical analyses, respectively) were adopted. The bonding adhesive was not explicitly modelled but was considered as explained later. The mechanical contribution of the fire insulation system was not considered, since it was estimated to be negligible.

The temperature-dependency of the properties of all constituent materials was incorporated in the models due its significant influence on the thermal and structural responses of the beams when subjected to fire. For the concrete a classical damaged plasticity model was adopted; a perfect elasto-plastic model was adopted for the steel rebars; and the CFRP strips were considered as linear elastic isotropic. A detailed description of the variation with temperature of the mechanical and thermo-physical properties is available in Firmo et al. (2015a). The CFRP-concrete bond was simulated by bi-linear global laws (cf. Figure 6a), which were independently determined in a previous investigation.
(Arruda et al. 2016) based on an inverse analysis of CFRP-concrete bond tests for temperatures up to 150 °C; such laws were calibrated in order to minimize the relative differences between numerical and experimental results obtained from double-lap shear tests (the test specimens were manufactured using the same materials that constitute the beams simulated in the present study). For temperatures above 150 °C, no CFRP-concrete interaction was considered in the models. Regarding the steel-concrete interaction, a perfect bond was assumed.

The FE models simulated the fire behaviour of the tested beams by means of a sequentially uncoupled thermo-mechanical procedure: the heat transfer analysis (with a maximum time step of 10 seconds) was first performed to obtain the temperature distributions for a duration similar to that observed in the tests; it was followed by a mechanical analysis which considered the thermal data determined in the previous step. It is worth pointing out that the same FE mesh was used in both steps. The numerical procedure adopted in the present study has shown to be able to predict the fire behaviour of insulated EBR-CFRP-strengthened RC beams (e.g. Dai et al. 2015 and Firmo et al. 2015a).

Figure 5 a) Comparison between the fire performance of EBR and NSM strengthening systems (experimental results); b) FE mesh of the beam’s models (example of model 50-25).

Numerical results

Figure 3a shows the comparison between predicted and measured temperatures along the bonded interface when the CFRP strengthening system lost its structural effectiveness due to the debonding from the beams’ soffit, which was clearly detected in both experimental and numerical midspan displacement vs. time curves showed in Figure 4a. Overall, the numerical curves presented in Figure 3a exhibited a similar behaviour to those measured during the fire resistance tests, although the FE models overestimated the temperatures along the bonded interface at the instant of CFRP debonding in beams 0-0 and 50-25 (this is clearly seen in Figure 3b). Such differences are related to the fact that the temperature distributions in these two beams had been computed for slightly longer periods of fire exposure – as observed in Figure 4b the numerical models over-estimated the fire resistance of the strengthening system in these beams (18 min vs. 10 min in beam 0-0 and 90 min vs. 82 min in beam 50-25).

Regarding the midspan displacement increase, the numerical curves presented in Figure 4a exhibited slightly higher deflection increase rates than those measured during the tests, which may stem from (i) some horizontal friction in the pinned supports (not considered in the FE models), and (ii) differences between the actual temperature dependencies of the materials’ mechanical properties and those considered in the models. Nevertheless, as depicted in Figure 4b the FE models were able to predict the loss of the CFRP system with reasonable accuracy (namely, if considering the complexity of the fire behaviour of NSM-strengthened members).

Figure 6b shows the average bond stress at the CFRP-concrete interface (computed along the CFRP anchorage length and along the central length) as a function of the time of fire exposure. For time = 0 min, as expected, the numerical results showed (i) similar bond stresses in all beams (considering the same location), and (ii) higher average bond stresses along the anchorage length of the strengthening system (La, cf. Figure 1) compared to those along the central length (span). Moreover, this figure shows that the average bond stresses in the anchorage and central zones were approximately constant during the initial stages of fire (in some cases presenting slight increases), after which those along the central length decreased to zero, whereas those along the CFRP anchorage length increased at a high rate, then presented a less pronounced increase and finally decreased to zero, with such instant matching the loss of the CFRP system (cf. Figure 4). This behaviour is more pronounced in beams 25-0 and 50-25, confirming that the thicker insulation boards applied in the CFRP extremities promote bond stress transfer from the central part of the strengthening system to the anchorage zones, allowing the CFRP strip to retain its structural effectiveness through a cable mechanism. These numerical results, besides providing further data that simply were not possible to quantify experimentally, validate the fire protection methodology adopted in the fire resistance tests.
Figure 6 a) Shear stress vs. slip curves for different temperatures (Arruda et al. 2016); b) average bond stress at the CFRP-adhesive interface (along the CFRP anchorage length and central zone) vs. time of fire exposure.

CONCLUSIONS

This paper presents experimental and numerical investigations about the fire behaviour of RC beams flexurally strengthened with NSM-CFRP strips when subjected to a sustained load and the standard ISO 834 fire. The test results confirmed the susceptibility of NSM-CFRP strengthening systems when exposed to fire - the mechanical contribution of the CFRP strips in a non-insulated beam was lost after only 18 minutes of fire exposure. However, the structural effectiveness of the CFRP system was considerably extended (up to 114 min) by applying a thin insulation layer along its central length and a thicker one (up to 50 mm) at both anchorage zones. With these fire protection schemes, although the CFRP-concrete bond became highly damaged along the central zone (less insulated), the CFRP strips continued to carry tensile forces as suspended “cables” fixed in the cooler (more insulated) anchorages zones. For the beams tested and the insulation schemes adopted, the maximum/critical temperature in the adhesive at the anchorage zones when the CFRP debonded attained values considerably higher than its $T_c$ (ranging from $2.4 \times T_c$ to $5.0 \times T_c$), indicating the over-conservatism of adopting it as a threshold for fire design in this particular case. Additionally, the results obtained also confirmed the much better fire resistance performance of NSM compared to EBR. Regarding the numerical investigations, the results obtained provided further insights about the stress transfer mechanism in NSM-CFRP strengthening systems during fire, confirming that the application of a thicker insulation layer in the CFRP anchorage zones allows making use of the cable behaviour and therefore extend the fire resistance of the strengthening system.

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Organizers:
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DESIGN MODELS FOR THE STRENGTH, THE STIFFNESS AND THE CYCLIC DEFORMATION CAPACITY OF RC MEMBERS RETROFITTED WITH FRP

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ABSTRACT

The idea of wrapping concrete in Fiber-Reinforced Polymer (FRP) to enhance its compressive strength and deformation capacity first appeared in the technical literature and was demonstrated experimentally more than 35 years ago, but it took some time for the industry and the research community to recognize its potential. At present its most common practical application in the seismic regions of the world is in upgrading the ductility and flexural deformation capacity of deficient concrete columns. In the first European Standard for the seismic retrofitting of existing buildings, i.e., Part 3 of Eurocode 8, an effective mechanical ratio of FRP is added to the corresponding ratio of confining steel, to take into account the beneficial effect of confinement by FRP on the cyclic ultimate chord rotation of concrete members having their end regions wrapped in FRP; the effective elastic stiffness of the member and its moment resistance are presumed, though, to be unaffected by the FRP. If longitudinal bars are lap-spliced along a length which is less than certain limit values, these properties, as well as the cyclic ultimate chord rotation, are taken to depend on the FRP. In the light of recent tests all these rules, proposed about ten years ago by the author's team, have certain bias and lack of fit. So, in view of the upcoming revision of the European Standard, the team has updated them, to reflect our current experimental knowledge.

KEYWORDS

Confinement, deformation capacity, Eurocode 8, FRP, lap-splices, RC columns, stiffness, ultimate deformation.

INTRODUCTION: FROM THE FIRST IDEA TO THE PRESENT DAY SEISMIC DESIGN CODES

The first idea to enhance the strength and deformation capacity of columns through confinement by FRPs (Fardis and Khalili, 1981, 1982) went unnoticed by researchers, but was taken up few years later by the construction industry. The classic book Teng et al (2004) notes: "The compressive strength enhancement of concrete due to the external wrapping of FRP was first demonstrated by Fardis and Khalili (1981, 1982). This concept was first applied to the strengthening of real RC columns in Japan in the mid 1980s." According to "... a learned group of writers, recruited from the ranks of the editorial board of the ASCE Journal of Composites for Construction " in their ASCE 150th Anniversary paper Bakis et al (2002): "Historically, composites were first applied as flexural strengthening materials for RC bridges (Meier 1987; Rostasy 1987) and as confining reinforcement of RC columns (Fardis and Khalili 1981; Katsumata et al. 1987). Developments since the first research efforts in the mid-1980s have been tremendous. .... The number of applications involving composites as strengthening/repair or retrofit materials worldwide has grown from just a few 10 years ago to several thousand today.” Indeed, among the over hundred references of Bakis et al (2002) only Fardis and Khalili (1981) date before 1987. According to Akovali (2002), "In recent years the use of FRP composites for the strengthening of concrete columns has become increasingly popular among engineers and researchers. ... Enhancement in the compressive strength of concrete as the result of external wrapping by the FRP was first demonstrated by Fardis and Khalili (1981, 1982)." Zhu et al (2005) opens as follows: "Early attempts by Fardis and Khalili (1981) showed great potentials for concrete-filled fiber reinforced polymer (FRP) tube (CFFT) as structural members. The concept was revitalized in the 1990’s as a composite structure with superior performance in axial compression (Mirmiran and Shahawy 1995). Since then, significant research has been carried out towards better understanding of the behavior of CFFT stubs and confinement modeling for FRP (Lorenzis 2001; Teng et al. 2003)". Chen et al (2013) states: "FRP composites with unidirectional fibers are commonly used in column strengthening with the fibers oriented in the direction perpendicular to the longitudinal axis of the column. The first experiments on this technique were conducted in the early 1980s (e.g., Fardis and Khalili 1981, 1982), and the concept has been investigated extensively since then (e.g., Ahmad et al. 1991; Karbhari et al. 1997; Rochette and Labossiere 2000; Tamuzs et al. 2007; Xiao and Wu 2000; De Luca et al. 2011)."
Another concept first proposed and demonstrated by Fardis and Khalili (1981, 1982) and rediscovered about 15 years later is summarized by Li et al (2005) “Recently, attention has been paid to new construction using FRPs, directly. The main aim in this new type of composite structures is to replace the steel reinforcements in concrete columns by laminated FRP shells, or exoskeletons, in order to achieve superior strength, ductility, and durability that are essential for concrete columns subjected to static and dynamic loads. This results in FRP tube-encased concrete columns (FRP/ECCs). A number of studies have been conducted on FRP/ECCs. In the early 1980s, the concept of FRP/ECC was proposed by Fardis and Khalili (1981, 1982), who analyzed the behavior of a circular FRP/ECC and a rectangular FRP tube-encased concrete beam. These structures have shown several advantages over the traditional RC columns. They have demonstrated an increase in the compressive strength and maximum moment, and a reduction in structural weight. They have also shown that an FRP tube directly serves as formwork and hence reduces the cost, increases the speed of construction, and improves the durability and watertightness. However, owing to the high initial cost associated with FRP tubes, little attention has been paid to this new concept. With the reduced cost of FRP tubes and with the success of FRP-repaired concrete columns, attention has been paid to FRP/ECCs in recent years. A new FRP tube system was proposed by Mirmiran (1996), Seible et al (1997) and Mirmiran and Shahawy (1997). In their works, the tube was a multilayer composite shell that consisted of at least two plies: an inner ply of axial fibers, and an outer ply of hoop fibers. It possessed the same advantages as that of the FRP tube proposed by Fardis and Khalili. The structural behavior of FRP/ECCs has been a research focus for the past five to ten years”. On the same topic Fam and Mirmiran (2013) write in their review of "Hybrid FRP/Concrete Rectangular Beam Sections": "One of the earliest attempts to produce hybrid FRP/concrete beam elements was done by Fardis and Khalili (1981) as they proposed casting concrete into FRP boxes. They also pointed out the mechanical role of FRP and concrete as follows: (1) FRP carries the tensile forces in the tension zone; (2) It provides partial confinement of concrete in the compression zone, enhancing strength and ductility; (3) It carries part of the shear force in the beam through the two sides; and (4) The concrete core provides compressive strength and rigidity and prevents local buckling of the FRP casing. They also pointed out that adhesion between the concrete and FRP is not necessary provided that the FRP box is closed at the two ends and the unidirectional fibers at the bottom have adequate end anchorage, otherwise mechanical interlock can be introduced. .... The strength of all the beams tested was greater than that of typical conventional reinforced concrete beams of the same dimensions and ranging from over-reinforced to overly under-reinforced. This study proved that FRP-encased concrete beams are superior to reinforced concrete beams on a strength/material cost basis.”

On the basis of their own concentric compression tests of nine cylindrical concrete specimens wrapped in GFRP, Fardis and Khalili (1981, 1982) proposed a hyperbola for the full stress-strain law of FRP-confined concrete. Its three parameters were defined by the initial tangent Modulus of concrete (taken to be unaffected by confinement), the ultimate strength (for which the Newman and Newman (1971) triaxial strength model was adopted) and the strain at ultimate strength (taken to be increased by 0.0005Epdferc, where £t and p are the FRP Elastic Modulus and geometric ratio in the circumferential direction and f, the uniaxial concrete strength). This law was used then to construct moment-axial load interaction diagrams for concrete columns wrapped in FRP.

The other important contribution of the writer to the application of FRPs in concrete construction came with the first European Standard for seismic retrofitting of existing buildings (CEN 2005), which incorporated expressions by Biskinis and Fardis (2006, 2010a, 2010b) for the ultimate chord rotation under cyclic loading and the secant-to-yield-point stiffness of concrete members, before and after retrofitting with externally applied FRP. As shown in Figure 1, this stiffness is taken as effective elastic stiffness. The ultimate deformation (chord rotation, curvature, etc) is commonly defined as the deformation after ultimate strength where lateral force resistance drops to less than 80% of the ultimate strength.

![Figure 1 Idealized moment-chord rotation relationship of concrete members](image)

**CURRENT MODELS IN EUROCODE 8 FOR THE YIELD MOMENT, THE STIFFNESS AND THE CYCLIC CHORD ROTATION CAPACITY OF FRP WRAPPED CONCRETE COLUMNS**

As explained in Fardis (2006) and Biskinis and Fardis (2006, 2007), Part 3 of Eurocode 8 (CEN 2005) takes the
effective elastic stiffness to be unaffected by FRP wrapping. It takes the ultimate cyclic chord rotation after Biskinis and Fardis (2006, 2010b), i.e., to be proportional to 25 to a power equal to the mechanical ratio of confining reinforcement in the direction of loading to times a theoretical effectiveness factor (cf. Eq. 10); for FRP-wrapped columns the exponent is increased by the mechanical ratio of FRP in the direction of loading, \( \rho f_{\text{eff}} f_c \), times a theoretical effectiveness factor:

\[
a_t = 1 - \frac{(h-2R)^2 + (b-2R)^2}{3bh}
\]

where \( R \) is the radius of the (chamfered) corners of the section and \( b, h \) the sides of its circumscribed rectangle (equal to 2\( R \) in circular columns). Instead of its ultimate tensile strength, \( f_{u,t} \), an effective FRP strength is used:

\[
f_{t,e} = \min \left( f_{u,t} - \frac{\rho f_{\text{eff}} E_t}{E_t} \right) \min \left( 0.5, \left[ 1 - 0.7 \frac{\rho f_{\text{eff}} E_t}{f_c} \right] \right)
\]

The limit strain, \( \varepsilon_{u,t} \), is 1.5% for Carbon FRP (CFRP) or Aramid FRP (AFRP), and 2% for Glass FRP (GFRP). If longitudinal bars, of diameter \( d_b \), are lap-spliced under the FRP-wrapping with a lapping, \( l_o \), shorter than:

\[
l_{o,\text{ma}} = 0.2 d_b \sqrt{f_{y,c}} \left( f_{y,c} \text{ in MPa}\right)
\]

the tension bars have their yield stress multiplied by \( l_o / l_{o,y,\text{ma}} \) whereas both lapped compression bars count in the compression reinforcement. Moreover, the plastic part of the cyclic ultimate chord rotation of a member with \( n_{\text{tot}} \) pairs of lapped bars around the section, is multiplied by \( l_o / l_{o,y,\text{ma}} \), if \( l_o \) is less than:

\[
l_{o,\text{ma}} = \frac{d_b f_{y,c}}{1.05 + 14.5 f_{y,c} / f_{c}} \left( f_{y,c} \text{ in MPa}\right)
\]

As shown in Figure 2, in the light of more recent tests the rules in (CEN 2005) summarized above suffer from a certain lack of fit and bias and need revisiting and updating. As the revision of this European Standard is underway, this paper revisits the current expressions and rules and makes proposals that better reflect the S-o-A.

![Figure 2](image-url)

Figure 2 Tests on FRP-wrapped rectangular columns vs predictions per (CEN 2005): (a), (d) yield moment; (b), (e) chord rotation at yielding; (c), (f) ultimate chord rotation; (a)-(c) continuous bars; (d)-(e) lap-spliced bars.

[Note: coefficient of variation (CoV) of test-to-prediction ratio: (a) 19.4%; (b) for undamaged columns 37.4% (median 1.03); (c) 30.6%; (d) 10.5%; (e) 17.7%; (f) 36.3%; median and CoV for the effective elastic stiffness \( EI_{\text{eff}} = M_L / 3 \theta_0 \) of undamaged columns: 1.02 and 30%].

**RECENT MODELS FOR THE YIELD MOMENT, THE EFFECTIVE STIFFNESS AND THE CYCLIC ULTIMATE CHORD ROTATION OF FRP WRAPPED COLUMNS WITH CONTINUOUS BARS**
Part 3 of Eurocode 8 (CEN 2005) applies to buildings alone; so, it covers only rectangular columns. As it neglects the effect of concrete strength enhancement by the FRP wrapping on the yield moment, $M_y$, and the secant-to-yield-point stiffness, $E_{AM}=M_y/L_e/3\phi_y$, it underestimates these properties (Figure 2). Neglecting the effect of concrete strength enhancement by the FRP leads to similar underestimation of these properties in circular columns (Biskinis and Fardis 2013a). Figure 3 shows that section analysis gives an unbiased estimate of the yield moment both for rectangular and for circular columns, if strength enhancement due to the FRP is accounted for, e.g., per Lam and Teng (2003, 2009), Jiang and Teng (2007), Teng et al (2009). The same analysis gives the yield curvature, $\theta_y$, from which the chord rotation of the shear span, $\theta_c$, at yielding, $\theta_c$, is obtained as follows (Biskinis and Fardis 2013a, 2013b):

- For rectangular columns:

$$\theta_y = \phi_y \left( L_e + a_d z \right) + 0.0014 \left( 1 + 1.5 \frac{h}{L_e} \right) + a_d \frac{df}{8f_e} \left( f_y, f_c: \text{MPa} \right)$$  \hspace{1cm} (4a)

- For circular columns:

$$\theta_y = \phi_y \left( L_e + a_d z \right) + 0.0027 \left( 1 - \min \left( 1; \frac{L_e}{7.5D} \right) \right) + a_d \frac{df}{8f_e} \left( f_y, f_c: \text{MPa} \right)$$  \hspace{1cm} (4b)

In Eqs. 4 $h$ is the member depth and $D$ the diameter of circular sections. The internal lever arm, $z$, is included in the first term if diagonal cracks form before the end section yields; i.e., if $V_c=M/L_e$ exceeds the shear resistance without shear reinforcement, then $a_c=1$; if it doesn’t, $a_c=0$. The last term is the fixed end rotation due to slippage of the tension bars - with diameter $d_b$ and yield stress $f_y$(MPa) - from their anchorage zone beyond the member end; it is omitted if symmetry or physical constraints suppress it ($a_d=0$); normally they don’t and then $a_d=1$.

The ultimate chord rotation at a member end, $\Delta \theta_u$, is the sum of the chord rotation at apparent yielding, $\Delta \theta_u^p$, and of a plastic part $\Delta \theta_u^d$. In the reference model in (Biskinis and Fardis 2013a, 2013b) $\Delta \theta_u^d$ was set equal to the sum of the chord rotation produced by the plastic part of the ultimate curvature, $\phi_y-\phi_y$, within a plastic-hinge length, $L_{ph}$, and the post-elastic fixed-end-rotation due to slippage of longitudinal bars from their anchorage beyond the member end, $\Delta \theta_{u,\text{slip}}$:

$$\Delta \theta_u = \Delta \theta_u^p + \left( \phi_y - \phi_y \right) \left( 1 - 0.5L_{ph}/L_e \right) + a_d \Delta \theta_{u,\text{slip}}$$  \hspace{1cm} (5)

with $a_d$ defined as for Eq. 4. Biskinis and Fardis (2010b) presented a section analysis for the ultimate curvature, $\phi_y$, of RC members without FRP wrapping, with equations and ultimate strain criteria for all failure modes. They also fitted expressions for $\Delta \theta_{u,\text{slip}}$ to relative rotation measurements including fixed-end-rotation due to bar slippage from the anchorage zone under cyclic loading:

$$\Delta \theta_{u,\text{slip}} = 5.5d_f\phi_y$$, \hspace{1cm} or \hspace{1cm} $\Delta \theta_{u,\text{slip}} = 5d_f\left( \phi_y + \phi_y \right)$  \hspace{1cm} (6)

For elastic-perfectly plastic $\sigma$- strain law of steel, Biskinis and Fardis (2010b) found optimal agreement of the computed and the measured values of $\phi_y$ in about 225 tests of rectangular sections without FRP-wrapping which failed by steel rupture, if the limit strain, $\varepsilon_{u,\text{nom}}$, of the extreme tension bars is taken equal to $(3/8)\varepsilon_{u,\text{nom}}$, with $\varepsilon_{u,\text{nom}}$ denoting the uniform elongation of a steel bar at its ultimate strength. The $\sigma$- strain law of FRP-confined concrete is taken per Lam and Teng (2003): first parabolic, then linear up to $f_{cc}$, with $f_{cc}$ per Lam and Teng (2003). Recall that the model for $f_{cc}$ in Lam and Teng (2003) is accompanied by expressions for the ultimate strain of FRP-confined concrete, $\varepsilon_{u,\text{cc}}$, fitted to monotonic or cyclic tests in concentric compression. However, the ultimate curvatures of FRP-wrapped columns computed from these expressions do not agree with the test results of such columns in cyclic flexure. A much better fit was obtained in Biskinis and Fardis (2013a) by taking $\varepsilon_{u,\text{cc}}$ from the following modification of equations fitted in Biskinis and Fardis (2010b) to measured ultimate curvatures of rectangular columns without FRP-wraps which failed in cyclic flexure by concrete crushing:

$$\epsilon_{u,c} = 0.0035 + \left( \frac{10}{h(\text{mm})} \right)^2 + a f_c \min \left( 0.5 \frac{\rho_y (0.6 \varepsilon_{u,\text{f}}, E_t)}{f_{cc}}, \left( 1 - \min \left( 1; \frac{L_e}{7.5D} \right) \right) \frac{df}{8f_e} \left( f_y, f_c: \text{MPa} \right) \right)$$  \hspace{1cm} (7)

where $\alpha$ is given by Eq. 1 and $\rho_y, E_{cc}$ have been defined in Section 2; $\varepsilon_{u,\text{f}}$ is the failure strain of FRP. For CFRP, GFRP and polyacetal fiber (PAF) sheets $c_f$ is equal to 0.2 for rectangular sections (Biskinis and Fardis 2013a) and to 0.18 for circular ones (Biskinis and Fardis 2013b), dropping to 0.17 and 0.12 for AFRP. The original expressions by Biskinis and Fardis (2010b) have, instead of the FRP term (the last one at the right-hand-side of Eq. (7), its counterpart for confinement by steel ties with geometric ratio $\rho_y$ and yield stress $f_{yw}$, namely $0.4\rho_y f_{yw}/f_{cc}$, where $\alpha$ is the effectiveness factor for confinement by ties. If the FRP produces less confinement than the ties, the column may survive rupture of the FRP and reach an ultimate curvature controlled by the confined concrete core inside the ties. This curvature is computed from section analysis with a parabolic-rectangular $\sigma$- strain law for the steel-confined concrete core, with $f_{cc}$ and the associated strain taken from (fib 2012):

$$f_{cc} = f_y (1 + K), \text{ with } K = 3.5 \left( \frac{\rho_y f_{yw}}{f_y} \right)^{3/4}$$, $\epsilon_{u,\text{cc}} = \epsilon_{u,c} \left[ 1 + \frac{f_{cc}}{f_y} - 1 \right]  \hspace{1cm} (8)$

and the concrete strain at $f_{cc}, \epsilon_{u,cc}$, taken as 0.002. The limit value, $\epsilon_{u,\text{cc}}$, at the end of the horizontal $\sigma$-branch is given by the original form of Eq. (7), i.e., with the last term replaced by $0.4\rho_y f_{yw}/f_{cc}$.
For so-determined \( \phi_h \), \( \phi_l \) and \( \Delta \theta_{\text{clip}} \), the expressions adopted in Biskinis and Fardis (2013a, 2013b) for the plastic hinge length, \( L_p \), are those fitted in Biskinis and Fardis (2010b, 2013b) to over 1500 tests of flexure-controlled rectangular members without FRP-wrapping and adopted in (fib 2012). These expressions are:

- For non-circular sections: 
  \[
  L_{pl} = 0.2h \left[ 1 + \frac{1}{3} \min \left( 9; \frac{L_t}{h} \right) \right] 
  \]  

- For circular sections: 
  \[
  L_{pl} = 0.6D \left[ 1 + \frac{1}{6} \min \left( 9; \frac{L_t}{D} \right) \right] 
  \]

![Figure 3 Tests of FRP-wrapped columns v predictions of (Biskinis and Fardis 2013a, 2013b): (a)-(c) rectangular columns with continuous bars; (d)-(f) rectangular columns with lap-spliced bars; (g)-(i) circular columns; (a), (d), (g): yield moment; (b), (e), (h): chord rotation at yielding; (c), (f), (i) ultimate chord rotation.]

[Note: CoV of test-to-prediction ratio: (a) 19.3%; (b) 37.3% (median: 1.015); (c) 30.4%; (d) 10.5%; (e) 17.7%; (f) 23.2%; (g) for continuous bars 22.5%, for lap-spliced bars 13.8% (medians: 1.005 and 0.89); (h) for continuous bars 22.3%, for lap-spliced bars 32.8% (medians: 1.005 and 0.93); (i) for continuous bars 22.4%, for lap-spliced bars 39.1% (medians 1.005 and 0.975); median and CoV of effective elastic stiffness: \( E_{eff} = M/L/3\theta \) of rectangular columns: 0.985 and 30.4% for continuous bars, 1.025 and 19.8% for lap-spliced ones; ibid for circular columns: 1.21 and 23.2% for continuous bars, 0.975 and 20.7% for lap-spliced bars].

Eqs. 1 and 4-9, used with the Lam and Teng (2003) strength model, give predictions which compare to tests as shown in Figure 3(a), (b) and (g)-(i). Medians and coefficients of variation (CoV) of the corresponding test-to-prediction ratio are given in Figure 3 and its caption. Those of the ratio of the experimental ultimate chord rotations of FRP-wrapped members with rectangular section to the outcome of Eqs. 1 and 4-9 (omitted from Figure 3) are equal to 0.99 and 32.7%, respectively. The comparison in Figure 3(c) is for a second option for rectangular FRP-
wrapped columns: an extension of an empirical model fitted by Biskinis and Fardis (2010b) to 1400 flexure-controlled tests of members without FRP-wrapping - adopted in (CEN 2005) and (fib 2012):

$$\theta_u = \theta_y + \theta^\text{pl}_u = \theta_y + 0.0145(0.25)^{\left(\frac{\max(0.01, \omega)}{\max(0.01, \omega_{\text{col}} - \omega)}\right)^{0.3}} \left(f_c(MPa)\right)^{0.35} \frac{L_y}{h}$$

(10)

where:
- \(\nu=N/bhf_e\), with \(b\) the width of the compression zone and the axial force \(N\) positive for compression,
- \(\omega_{\text{col}}=\rho_{\text{col}}/f_c\) and \(\omega^d=\rho_{\text{f}}/f_c\) total and compression mechanical reinforcement ratio of the section,
- \((a\varphi/f_c)\); confinement term due to the FRP, given by Eq. 11 below (with \(f\) from Eq. 1 and \(c=1.8\) for CFRP or PAF sheets and \(c=0.8\) for GFRP or AFRP) instead of the current term in (CEN 2005) given by Eq. 3.

$$a_{\varphi}/f_c = a_f \beta_f \min\left[0.4; \frac{0.6\varepsilon_{f} E_f}{f_c}\right] + \frac{0.6\varepsilon_{f} E_f}{f_c} \left[1-0.5\min\left[0.5; \frac{0.6\varepsilon_{f} E_f}{f_c}\right]\right]$$

(11)

The test-to-prediction ratio of Eqs. 10 and 11 (cf Figure 3(c)) has slightly better statistics than that of Eqs. 4-9.

More recently Grammatikou et al (2016a, 2016b, 2017) used a large databank of curvature measurements in tests that led to flexure-controlled ultimate conditions to fit more elaborate ultimate strain criteria for circular sections and sections with rectangular or triangular compression zone. They proposed the following ultimate strains for unconfined and FRP-confined concrete, respectively, in members subjected to cyclic flexure:

$$0.0035 \leq \varepsilon_{\text{cu}} = \left(18.51/h(\text{mm})\right)^2 \leq 0.01$$

(12)

$$\varepsilon_{\text{cu},\text{c}} = \varepsilon_{\text{cu}} + a_f \beta_f \min(0.5; \rho_f(0.6\varepsilon_{f} E_f)/f_c) \left[1 - \min(0.5; \rho_f(0.6\varepsilon_{f} E_f)/f_c)\right]$$

(13)

Coefficient \(a_f\) is given by Eq. 1; \(\beta_f\) is equal to 0.115 for CFRP, GFRP or PAF sheets and to 0.1 for AFRP.

FRP-wrapped members fail in cyclic loading mostly by steel rupture, but reach higher bar strains than members without FRP wrapping. The tension zone performs better inside an FRP-wrapping, because the FRP prevents the buckling of bars that precedes bar rupture in cyclic tests of members without FRPs. For this reason the strain of bars inside FRP wraps at the instant the ultimate curvature occurs in cyclic loading does not depend on the tie-spacing-to-bar-diameter ratio, nor on the number of longitudinal bars in the compression zone, as bar strains at ultimate curvature of members without FRP do. It is larger than the latter and - for probabilistic reasons explained for monotonic loading in (Grammatikou et al 2016a) - decreases with respect to the average bar strain at tensile strength in a standard bar test, \(\varepsilon_{\text{bar,nom}}\), as the number of bars in the tension zone, \(N_{\text{bar,tension}}\), increases:

$$\varepsilon_{\text{cu},\text{c}} = 0.6\varepsilon_{\text{cu,nom}} \sqrt{1-0.15\ln\left(N_{\text{bar,tension}}\right)}$$

(14)

Using an enriched experimental database, Grammatikou et al (2016a) also revised Eqs. 6 and 4 as follows:

$$\Delta\theta_{u,\text{slip}} = 4.5d_l \phi_u, \quad \text{or} \quad \Delta\theta_{u,\text{slip}} = 4.25d_l (\phi_u + \phi_f)$$

(15)

- For rectangular columns:

$$\theta_y = \phi_u \frac{L_y + a_f \varepsilon_{\text{f}}} {3} + 0.0015 \left[1 + \frac{0.355}{L_y} \frac{h}{L_y}\right] + a_f \frac{d_l f_f}{8f_c}, \quad (f_f, f_c: \text{MPa})$$

(16a)

- For circular columns:

$$\theta_y = \phi_u \frac{L_y + a_f \varepsilon_{\text{f}}} {3} + 0.0025 \left[1 - \frac{1}{2}\min\left(1; \frac{L_y}{8D}\right)\right] + a_f \frac{d_l f_f}{8f_c}, \quad (f_f, f_c: \text{MPa})$$

(16b)

Grammatikou et al (2016a) revisited \(L_y\) as well, fitting the following expressions to the measured ultimate chord rotations of over 2000 flexure-controlled reinforced concrete specimens without FRP in cyclic loading:

- For beams or columns with section of rectangular parts, walls of all types and hollow piers.

$$L_y = 0.5h \left(1 + 0.4 \left[\min\left(9, \frac{L_y}{h}\right)\right] \min\left(2.5, \max\left(0.05, \frac{b_w}{h}\right)\right) \left[1 - 0.45 \min(0.7, v)\right)\right]$$

(17a)

- For columns with circular section:

$$L_y = 0.7D \left(1 + \frac{1}{2} \min\left(9, \frac{L_y}{D}\right) \left[1 - \min(0.7, v)\right)\right]$$

(17b)

In Eq. 17a \(b_w\) is the width of one web (even in hollow or C-sections with more than one web); the axial force \(N\), positive for compression, is normalized to the cross-sectional area \(A_e\). as \(\nu=N/A_f\).

Figs. 4(a), (b) compare predictions of Eqs. 12-17 to test results of FRP-wrapped rectangular or circular columns respectively (Grammatikou et al 2016b); the associated CoV of the test-to-prediction ratio is 36.6% or 22.4%.

Grammatikou et al (2016b) also produced a set of alternatives to the empirical expression underlying Eq. 10 for members with a rectangular compression flange. These alternatives are extended for FRP-wrapping as follows:
\[ \theta_u = 0.0205 \left[ (1 - 0.4a_{uw}) (1 - 0.32a_{uw}) \right] 0.325^2 \left[ \frac{\max (0.01; \omega)}{\max (0.01; \omega_{uu} - \omega)} \right]^{0.2} \cdot \]

\[ f_r (MPa)^{0.15} \left[ \min \left( \frac{L_b}{h} \right) \right]^{0.35} 12.5 \left[ \frac{\omega_{u,exp}}{\omega_{u,pred}} \right] \]

\[ \theta_u = \theta_s + \theta_{u,p} = \theta_s + 0.0205 \left( (1 - 0.41a_{uw}) (1 - 0.31a_{uw}) \right) 0.2^{\gamma} \left[ \frac{\max (0.01; \omega)}{\max (0.01; \omega_{uu} - \omega)} \right]^{0.25} \cdot \]

\[ f_r (MPa)^{0.1} \left[ \min \left( \frac{L_b}{h} \right) \right]^{0.35} 24 \left[ \frac{\omega_{u,exp}}{\omega_{u,pred}} \right] \]

\[ \theta_u = \theta_s + \theta_{u,p} = \theta_s + 0.032 \left( 1 - \frac{1}{12} \max \left( 4 \min \left( \frac{a}{b_w} \right) \right) \right) 0.2^{\gamma} \left[ \frac{\max (0.01; \omega)}{\max (0.01; \omega_{uu} - \omega)} \right]^{0.275} \cdot \]

\[ f_r (MPa)^{0.1} \left[ \min \left( \frac{L_b}{h} \right) \right]^{0.35} 25 \left[ \frac{\omega_{u,exp}}{\omega_{u,pred}} \right] \]

\[ \theta_u = \theta_s + \Delta \theta_{u,slip} + \theta_{u,p} = \theta_s + \Delta \theta_{u,slip} + 0.013 \left( 1 - 0.27a_{uw} \right) 0.2^{\gamma} \left[ \frac{\max (0.01; \omega)}{\max (0.01; \omega_{uu} - \omega)} \right]^{0.275} \cdot \]

\[ f_r (MPa)^{0.1} \left[ \min \left( \frac{L_b}{h} \right) \right]^{0.35} 7.5 \left[ \frac{\omega_{u,exp}}{\omega_{u,pred}} \right] \]

\[ \theta_u = \theta_s + \Delta \theta_{u,slip} + \theta_{u,p} = \theta_s + \Delta \theta_{u,slip} + 0.0123 \left( 1 - 0.05 \max \left( 4 \min \left( \frac{a}{b_w} \right) \right) \right) 0.225^{\gamma} \]

\[ \left[ \frac{\max (0.01; \omega)}{\max (0.01; \omega_{uu} - \omega)} \right]^{0.175} f_r (MPa)^{0.15} \left[ \min \left( \frac{L_b}{h} \right) \right]^{0.45} 7 \left[ \frac{\omega_{u,exp}}{\omega_{u,pred}} \right] \]

The term for the confinement by FRP (the very last one appearing in exponent), is still given by Eq. 11, but with:

- \( c_f = 1.9 \) for CFRP or PAF, \( c_f = 0.9 \) for AFPRP, \( c_f = 1.15 \) for GFRP in Eqs. 18a-18c;
- \( c_f = 2.8 \) for CFRP or PAF, \( c_f = 0.95 \) for AFPRP, \( c_f = 1.15 \) for GFRP in Eqs. 18d and 18e.

Eqs. 18b-18e account separately for the chord rotation at yielding, from Eq. 16a; Eqs. 18d, 18e explicitly account for the fixed-end-rotation due to bar slippage from the anchorage zone, from Eq. 15. Eqs. 18b, 18e explain the effect of cross-sectional shape on ultimate chord rotation as one of slenderness of the web. However, the improved rationality from Eq. 18a to 18e is not translated into much better fit or accuracy: all five versions of Eq. 18 produce essentially the same median (about 1.0) and CoV (31% to 31.5%) of the test-to-prediction ratio.

![Graphs showing cyclic ultimate chord rotation of FRP-wrapped columns with continuous bars, test prediction (a), (b) Eqs 12-17; (c) Eq 18c; (a), (c): rectangular; (b) circular columns (Grammatikou et al 2016b).](attachment:graph.png)

[Note: CoV of test-to-prediction ratio: (a) 36.6%; (b) 22.4% (c) 31.5%].

### RECENT MODELS FOR THE YIELD MOMENT, THE EFFECTIVE STIFFNESS AND THE CYCLIC ULTIMATE CHORD ROTATION OF FRP WRAPPED COLUMNS WITH LAP-SPLICED BARS

For rectangular columns Biskinis and Fardis (2013a) modified Eq. 4 into Eq. 19 below. If \( l_s \) is less than \( l_{uu, min} \), the
plastic part of the ultimate chord rotation from the empirical model, Eq. 10, is multiplied by $l_ou/ou,\text{min}$.

$$
\psi_{\text{circ}} = \psi_{\text{rect}} \left( \frac{d_u f_y}{1.05 + 14.5 \left( \frac{2}{n_{\text{tension}}} \right) \left( \frac{ap_g su}{f_c} f_y \right)^{1/2}} \right) (f_y, f_u, f_c \text{ in MPa})
$$

(19)

with $(ap_g su, f_c)$ from Eq. 11, i.e., the same value as in the second term of the exponent of the confinement term (the last one) in Eq. 10. If $n_{\text{tension}}=2$ this second term in the exponent is neglected by Biskinis and Fardis (2013a).

Eq. 19 does not apply to FRP-wrapped circular columns with lap-spliced bars; for them Biskinis and Fardis (2013b) introduced a different expression:

$$
\psi_{\text{circ}} = \psi_{\text{rect}} \left( \frac{d_u f_y}{5 + 6 \left( \frac{ap_g su}{f_c} f_y \right)^{1/2}} \right) (f_y, f_u, f_c \text{ in MPa})
$$

(20)

with $(ap_g su, f_c)$ still from Eq. 11. Only the physical model of Eq. 5 is available for the ultimate chord rotation of circular columns. In that context Biskinis and Fardis (2013b) proposed to reduce the limit strain of lap-spliced bars according to Eq. 21, with $\epsilon_{ou,\text{min}}$ from Eq. 3, $l_{ou,\text{min}}$ from Eq. 20, and $\epsilon_{ou}$ equal to $(3/8)\epsilon_{ou,\text{nom}}$:

$$
\epsilon_{ou} = \left[ \frac{1.2 - \frac{l_o}{l_{ou,\text{min}}} - 0.2}{\epsilon_{ou}} \right] \epsilon_{ou} \geq \frac{l_o}{l_{ou,\text{min}}} \frac{f_y}{E_y}, \text{ if } l_o \leq l_{ou,\text{min}}
$$

(21)

The predictions of these rules per Biskinis and Fardis (2013a, 2013b) are compared in Figures 3(d)-(f) and 3(g)-(i) to the results on tests of rectangular and circular columns, respectively. The comparison is satisfactory for rectangular columns. For circular ones the yield moment, the chord rotation at yielding and - to a lesser extent - the ultimate chord rotation are overpredicted. In other words, rules mimicking those developed for rectangular columns with or without FRP wrapping do not suit well circular ones. Note also that, although for rectangular columns with FRP-wrapping and continuous bars the physical model of Eq. 5 comes out almost as accurate as the empirical one of Eq. 10, Biskinis and Fardis (2013a) could not extend it to FRP-wrapped rectangular columns with lap spliced bars in a way analogous to Eq. 21 - originally fitted to rectangular columns with lap splices but no FRP-wrapping.

Figure 5 Test results on FRP-wrapped columns with lap-spliced deformed bars v prediction of: (a), (d) yield moment; (b), (e) chord rotation at yielding; (c), (f) cyclic ultimate chord rotation; (a)-(c): rectangular; (b) circular columns (Grammatikou et al. 2017).

[Note: CoV of test-to-prediction ratio: (a) 18.9%; (b) 19%; (c) 35.1%; (d) 13.1%; (e) 25.4%; (f) 31%].

Grammatikou et al. (2016b) addressed shortcomings of the Biskinis and Fardis (2013a, 2013b) approach in a more rational model, which applies seamlessly to rectangular and circular columns, with or without FRP wraps over the lap-splice region. First, they replaced Eq. 3 with:

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where the concrete tensile strength, $f_{ct}$, is taken per (fib 2012) as $f_{ct}(MPa)=0.3(f_c(MPa))^{2/3}$ and $c_{min}$ is the minimum concrete cover of the lapped bars, or half the clear distance to the closest lap-spliced bar, whichever is smaller.

Second, they introduced a common rule for $l_{ou,\min}$ in rectangular and circular sections and for confinement of the lap splice by an external FRP jacket or by internal steel ties:

$$l_{ou,\min} = \frac{d_k f_y}{f_{ct} \max \left( c_{min} / d_k, 0.7 \right)}$$

(22)

where:

- $a_o$ is the confinement effectiveness factor: $a_o=n_{tied}/n_{tot}$ in rectangular sections with $n_{tied}$ restrained lapped bar pairs (eg. at a chamfered corner of a FRP jacket) out of $n_{tot}$ lapped bar pairs in total; in circular sections $a_o=1$;
- $a$ is the confining medium factor: $a=9.5$ for confinement by CFRP or PAF sheets, $a=10.5$ for confinement by GFRP and $a=12$ for AFPR; $a=7.5$ for confinement by steel ties.
- $R_c$ is the confining medium radius, i.e., the bending radius of the FRP jacket or steel tie confining the splice;
- $p_{c}$ is the confinement pressure on the lap splice, equal to $t f_{cd}/R_c$ for confinement by FRP of thickness $t_i$, or $A_{dfc}(s/R_c)$ for confinement by steel ties with cross-sectional area $A_{sh}$ and spacing $s_0$.
- $a_i$ is a factor for confinement effectiveness along the length of the member, equal to 1.0 for confinement by FRP; for confinement by ties, it is equal to:

$$a_i = \left( 1 - \frac{h_o}{b_o} \right) \left( 1 - \frac{s_k}{b_p} \right)$$

(24)

with $h_o, b_o$: dimensions of the confined core to the centerline of the outer tie ($h_o=b_o=D_A$ in a circular core).

If $L_c < l_{ou,\min}$, the ultimate chord rotation is computed from the physical model, Eqs. 5, 12-17, but with the ultimate strain of tension bars reduced with respect to the value $\varepsilon_{fu}$ from Eq. 14 for bars inside an FRP jacket.

$$\varepsilon_{su,lap} = \min \left( \varepsilon_{fu} l_o / l_{ou,\min} \right) \left( 1 - l_o / l_{ou,\min} \right) f_y / E_y$$

(25)

Figure 5 compares test results from columns with FRP wrapping of the lap-splice region to predictions as given by Grammatikou et al (2006b) and gives statistics of the test-to-prediction ratio.

CONCLUDING REMARKS

After recalling the first record in the international scientific and technical literature of concrete confinement by FRP wrapping and of concrete cast in permanent FRP moulds, the paper highlighted the recent and current contributions of the author and his team to the development of practical design models for the enhancement of the cyclic deformation capacity of concrete members by wrapping their end regions with FRP. The models are seamless extensions of those proposed by the same team for all sorts of members without FRP wraps, including members with short lap splices of the longitudinal bars. They are based on the largest available databank of cyclic test results on concrete members with or without retrofitting. Earlier versions of the models for the cyclic deformation capacity and the secant-to-yield-point stiffness have been adopted in the 2005 European Standard for seismic assessment and retrofitting of existing buildings, Part 3 of Eurocode 8. In view of the upcoming revision of this European Standard, the current, enriched database of test results has been utilized to improve, harmonize and expand the portfolio of design models for the flexural resistance, the secant-to-yield-point stiffness and the cyclic deformation capacity of concrete members with or without retrofitting.

Going back to the beginning of the story, Fardis and Khalili (1982) proposed to take the ultimate strain of concrete as increased by 0.0005$E_y / \rho f_c$, (where $E_y$ and $\rho f_c$ are the FRP Elastic Modulus and geometric ratio in the circumferential direction) to account for the beneficial effect of confinement by FRP. If this, admittedly very conservative value is used in lieu of Eq. 12, the median of the test-to-prediction ratio for the cyclic ultimate chord rotation increases from 1.00 to 1.26 for rectangular columns with continuous bars and to 1.81 for circular ones; for lap-spliced bars and rectangular columns it stays at 1.00 and increases to 1.04 for circular. In all these cases the Coefficient of Variation (CoV) of the test-to-prediction ratio is about the same as when Eq. 12 is used. The different sensitivity of these results to the ultimate strain of confined concrete is due to the different failure modes: in columns with short lap-splices, the tension bars are the critical link, whereas in columns with continuous bars the FRP-confined concrete normally governs.

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ABSTRACT

Buckling of steel reinforcement usually causes a sudden loss of the load-carrying capacity and leads to the ultimate state of conventional reinforced concrete (RC) columns. However, it behaves differently in fiber-reinforced polymer (FRP)-confined RC columns due to the lateral confinement effect of FRP. This paper presents a theoretical study into the buckling behavior of longitudinal steel reinforcing bars embedded in FRP-confined concrete subjected to cyclic axial compression. An empirical monotonic compressive stress-strain model for laterally supported reinforcing bars considering the buckling effect, which was proposed in the authors’ previous study, was extended to a cyclic stress-strain model following the Menegotto-Pinto model accounting for the cyclic loops. The cyclic stress-strain models for laterally supported reinforcing bars as well as FRP-confined plain concrete were then implemented into the OpenSees platform to simulate the cyclic compressive behaviour FRP-confined RC columns and validated through comparisons with test results. The proposed cyclic stress-strain model for laterally supported reinforcing bars can serve as a fundamental law for the modelling of seismic behaviour of FRP-strengthened RC columns, especially for those with sparsely-spaced transverse ties where buckling of reinforcing bars becomes a significant concern.

KEYWORDS

Stress-strain model, bar buckling, FRP confinement, cyclic model, OpenSees.

INTRODUCTION

Old reinforced concrete (RC) columns, particularly those built prior to the 1970s, often had poor transverse reinforcement details. Consequently, the longitudinal reinforcing bars may buckle easily under a critical level of compressive strain due to insufficient lateral support. Buckling of longitudinal reinforcing bars in conventional RC columns is a vital damage state indicating a significant loss of the load-carrying capacity of RC columns. The effectiveness of external FRP jackets in resisting the buckling of longitudinal reinforcing bars in FRP-confined RC columns has been reported by previous researchers (e.g., Priestley et al. 1996; Bournas and Triantafillou 2011). It is generally believed that FRP can prevent, or at least postpone the buckling of longitudinal reinforcing bars (Bournas and Triantafillou 2011). In addition, because the concrete cover is confined by the external FRP jacket in an FRP-confined RC column, compared to the progressive buckling of longitudinal reinforcing bars, the brittle fracture of the external FRP jacket usually becomes a more critical factor leading to the sudden loss of the member’s load-carrying capacity and deformability. However, so far the behavior of progressive buckling of reinforcing bars, which may occur before the rupture of external FRP jackets, remains unclear.

For an accurate prediction of the strength and especially the post-peak behavior of FRP-confined RC columns under seismic loading, it is important to understand the interaction mechanism between steel reinforcing bars and FRP jackets at all deformation levels. Recently, Bai (2014) conducted an experimental study on the buckling behavior of reinforcing bars in FRP-confined RC columns subjected to monotonic axial compression. In this study, two types of FRP materials, carbon FRP (CFRP) and polyethylene terephthalate (PET) FRP, were used to compare the buckling restraining effect of FRP jacket with different jacket stiffness. PET FRP, being a new type of composite material, is characterized with a low elastic modulus but large rupture strain (Anggawidjaja et al. 2006, Dai et al. 2011). Test results showed that significant buckling was observed in PET FRP-confined RC columns before the rupture of PET FRP jacket, while no visible buckling phenomenon was found in CFRP-confined RC columns. In other words, the use of PET FRP allows the progressive buckling to occur while the rupture of CFRP preceded the occurrence of steel buckling. Therefore, the development of reinforcing bar buckling depends
strongly on the stiffness and elongation of FRP jacket. To interpret the test results, Bai and Dai (2015) further developed a “beam-on-elastic-foundation” model to simulate the compressive behavior for laterally supported reinforcing bars in FRP-confined circular RC columns based on the finite element (FE) approach. In this model, the confining effect on the longitudinal reinforcing bars provided by the FRP-confined concrete is modeled by a series of linear springs, which stiffness was evaluated based on a curved beam model. Furthermore, based on parametric studies, an empirical monotonic stress-strain model for the reinforcing bars embedded in FRP-confined RC columns considering the buckling effects was developed. Along with these previous work, this study aims to develop a cyclic stress-strain model for reinforcing bars embedded in FRP-confined circular RC columns with due consideration of the interaction between the progressive buckling of reinforcing bars and the FRP confinement effect, to facilitate the analysis of the behavior of FRP-confined circular RC columns under seismic loading. The proposed model will be implemented into the OpenSees (2009) platform and verified its accuracy through comparisons with test results.

THEORETICAL STRESS-STRAIN MODEL FOR LATERALLY SUPPORTED REINFORCING BARS UNDER CYCLIC LOADING

Monotonic Compressive Stress-Strain Model

A monotonic stress-strain model considering the buckling effects for longitudinal reinforcing bars embedded in FRP-confined RC columns was developed by Bai (2014). Revisions were then made to reflect the additional effects of lateral spring supports besides those of the slenderness ratio and yield strength of steel material, which were already considered in Dhakal and Maekawa’s (2002) three-stage model for the buckling reinforcing bars with lateral confinement. A set of empirical formulas were proposed based on extensive least-squares regressions of numerical data generated by FE analyses of the stress-strain behavior of laterally supported reinforcing bars subjected to monotonic loading. The second stage of Dhakal and Maekawa’s (2002) model was modified to account for the effect of lateral FRP confinement, and the characteristic point ($\sigma^*$, $\epsilon^*$) that intersects the second and third stages and the slope of the descending stress-strain curve of the third stage were defined (Figure 1):

$$
\begin{align*}
\sigma_i &= \sigma_0 \epsilon_i, & \epsilon_i \leq \epsilon_s \\
\sigma_i &= \frac{1 - (1 - \frac{\sigma_s}{\sigma_f}) (\epsilon_i - \epsilon_s)}{\epsilon_i - \epsilon_s}, & \epsilon_s < \epsilon_i < \epsilon^* \\
\sigma_i &= \sigma^* - E_s (\epsilon_i - \epsilon^*) \geq 0.2f_y, & \epsilon_i > \epsilon^*
\end{align*}
$$

(1)

and

$$
\begin{align*}
\frac{\epsilon^*}{\epsilon_y} &= 0.95(1.0+6.53\lambda_1 + 1.75\lambda_2)(0.90 + 1.14\lambda_3)(4.14\lambda_4-2.41) \geq 7 \\
\frac{\sigma^*}{f_y} &= 0.74(1.0+0.76\lambda_1)(0.72 + 0.24\lambda_2)(0.40 + 0.63\lambda_3) \geq 0.2
\end{align*}
$$

(2)

where $\lambda_1$, $\lambda_2$ and $\lambda_3$ are variables representing the effects of spring stiffness $K$, the slenderness ratio $L/D$ and the yield strength $f_y$, respectively, as follows:

$$
\begin{align*}
\lambda_1 &= \sqrt{K}/100 \\
\lambda_2 &= 10D/L \\
\lambda_3 &= \sqrt{450/f_y}
\end{align*}
$$

(3)

The slope of the third stage $E_3$ varies slightly with the spring stiffness $K$, the slenderness ratio and yield strength of steel material and its value can be determined using the following empirical equation:

$$
1000\frac{E_3}{f_y} = 1.31(1.0 + 1.28\lambda_1)(4.39-0.47\lambda_2\lambda_4)
$$

(4)

where $E_s$ is the elastic modulus of steel material.
Cyclic Rules

For analysis of seismic performance of FRP-confined RC columns, a cyclic stress-strain model of reinforcing bars with hysteretic loops is needed. In this study, the proposed cyclic stress-strain model for laterally supported longitudinal reinforcing bars with buckling effects simply follows the cyclic rules adopted in Menegotto and Pinto (1973) (Figure 2). In addition, it combines the monotonic tensile stress-strain model (Mander 1983) and the proposed monotonic compressive stress-strain model, which can consider the buckling effects of reinforcing bars as mentioned in the previous section.

Figure 1 Monotonic stress-strain model for laterally supported reinforcing bars

Figure 2 Menegotto-Pinto model

MODEL VERIFICATION

Bai (2014) conducted cyclic compressive tests on 12 circular circular columns (200 mm in diameter and 500 mm in height), comprising 6 FRP-confined RC columns and 6 FRP-confined plain concrete columns. The test parameters included the type of FRP [i.e., PET FRP and CFRP], the number of plies (layers) in the FRP jacket (i.e., one ply and two plies). Here only the cyclic compressive tests on two RC columns confined with one ply and two plies of PET FRP sheets (i.e., specimens PET-RC-1-c-a and PET-RC-2-c-a) were chosen for verification of the proposed cyclic stress-strain model for reinforcing bars considering the buckling effect. The OpenSees (2009) platform was used to simulate the two PET FRP-confined RC columns. The proposed cyclic stress-strain model for rebar with buckling effects was implanted into OpenSees as a new material and named “ReinforcingSteelBai”. The “NonlinearBeamColumn” element available in OpenSees, which is a force-based nonlinear beam-column element and considers the spread of plasticity, was adopted to simulate the reinforcing bars. Three Gauss-Lobatto integration points were defined along each element in the model (Figure 3).

Figure 3 Numerical model for FRP-confined RC column

The numerical simulation was conducted with the displacement control method. Unloading/reloading cycles were initiated at the same axial displacements as recorded in the tests. The analyses were terminated when the ultimate strain of FRP-confined concrete was reached. Figs.4a and 4b present the predicted axial load-displacement curves of cyclically loaded FRP-confined RC columns in comparison with the test results. For comparison purpose, there are two predicted load-displacement curves in each figure: one was based on the proposed cyclic stress-strain models of laterally supported reinforcing bars while the other was based on the theoretical cyclic stress-strain model for a bare bar without any consideration of buckling effect. It is seen that better agreement between the test results and FE predictions can be achieved when the buckling effect of laterally supported reinforcing bars is taken into account, which verifies the reliability of the proposed cyclic stress-strain model.
CONCLUSIONS

A cyclic stress-strain model for laterally supported reinforcing bars has been developed based on the combinations of the following three models: (1) monotonic compressive stress-strain model for laterally supported reinforcing bars including the buckling effect; (2) monotonic tensile stress-strain model for steel material; (3) Menegotto and Pinto model for the cyclic loops. The proposed cyclic stress-strain model for laterally supported reinforcing bars has been implemented into the OpenSees software platform. A simplified cyclic stress-strain model for FRP-confined plain concrete was also incorporated. The developed program was then used to simulate the load-displacement responses of FRP-confined RC columns under cyclic axial compression. Comparisons between test results and model predictions verified the accuracy of the proposed cyclic model for laterally supported reinforcing bars and demonstrated the significance of considering the bulking effects of reinforcing bars in the simulation.

ACKNOWLEDGMENT

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REFERENCES

SEISMIC RETROFITTING OF REINFORCED CONCRETE STRUCTURES USING COMPOSITES: FRENCH GUIDELINE

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ABSTRACT

In various countries, including France, the regulations concerning buildings and civil engineering structures contain recommendations aimed at achieving acceptable seismic performance, that is, the structures designed must withstand minor earthquakes without damage, moderate earthquakes with minimum non-structural damage and major earthquakes without collapsing. The seismic recommendations proposed in building regulations have thus been updated over the years to achieve this aim. In France, the new earthquake zone map and changes in the regulations as a result of Eurocode 8 (EC 8) have contributed to defining the performance objectives of new structures. For existing structures, at least in certain cases, reinforcement is required to reduce seismic risks. This notion is introduced in Eurocode 8 Part 3 and in the implementing decrees and orders. Seismic retrofitting can therefore be either voluntary or compulsory. Based on the current construction rate, it would take one hundred years to completely replace France's housing stock. Seismic retrofitting of existing structures therefore appears necessary to ensure the solidity of all building constructions and the safety and security of people and property. The need to reduce the effects of an earthquake on a reinforced concrete structure can correspond to two different situations:
1. the weak points identified are localised and confined within a particular area
2. there are many weak points everywhere and a global approach is necessary

In the first case, the strategy will consist in treating the problem of a single component whose probable failure would lead to consequences on a global level. In the second case, the strategy will consist in carrying out major works on the entire structure. In both cases, the retrofitting works will need to be validated on both a technical and economic level. Several reinforcement scenarios are possible: reduction in seismic loads (insulators, dissipators, change of mass and/or improvement of performance, resistance and/or rigidity of structure).

The retrofitting techniques generally used are classified according to purpose and technology:
- Retrofitting by addition (chaining, bracing, buttressing, etc.)
- Enhancement of shock absorption and/or reduction in rigidity (isolators, dissipators, etc.)
- Enhancement of strength and/or ductility (sprayed concrete, bonded composites, steel lining and bracing, etc.)
- Anchoring (floor-chaining, framework-chaining, foundation-framework)

This document presents recommendations for the seismic retrofitting of concrete structures using fibre-reinforced polymers (FRP).

KEYWORDS

Seismic retrofitting FRP, RC beams, guideline.

INTRODUCTION

For more than fifteen years, national and international research (U. Meier [1], K. Neale [2], P. Hamelin [3], E. Ferrier [4], A. Triantafillou [5], J-L. Clément [6]), has shown that composite materials (polymer Matrix, textile reinforcements) can be used to protect, repair and reinforce not only reinforced concrete, prestressed concrete,
metal and timber structures but also masonry structural components. The technologies used are of three types (wet lay-ups, adhesive bonding of flat pultruded composites and bag moulding) and the main materials concerned are carbon, glass and aramid fibres combined with epoxy-type thermosetting Matrix.

The performance of the retrofitted system is essentially related to the load transfer between the reinforcement and its substrate, by means of adhesive bonding or confinement.

Research results have been used to draw up recommendations and design rules in the case of quasi-static loading, taking serviceability limit state and ultimate limit state conditions into consideration (ACI 440 [7], ISIS Canada [8], JCI Japan [9], AFGC [10], Fib TG9.3 [11], EC8–3).

The aim of this document is to provide in-depth information on the use of materials for the seismic retrofitting of existing structures. These guidelines are a complement to existing guidelines on earthquake engineering.

Figure 1 is a diagrammatic representation of how earthquake risk prevention regulations are organised (on the date on which the present recommendations were finalised).

The main regulations defining earthquake building regulations are:

- the environment code, particularly articles R.563-1 to R.563-8 recently updated by:
- decree n°2010-1254 of 22 October 2010 relating to earthquake risk prevention
- decree n°2010-1255 of 22 October 2010 delimiting earthquake zones on French territory
- the order of 22 October 2010 amended relating to classification and seismic construction regulations applicable to buildings in the "under no special risks" class,
- the order of 4 October 2010 concerning special risks
- the order of 24 January 2011 concerning installations requiring an environmental permit
- the order of 26 October 2011 applicable to bridges "under no special risks"

N.B. ICPE: Installations Classées pour la Protection de l’Environnement (installations requiring an environmental permit)

Figure 1 Organisational structure of earthquake risk prevention regulations [12]

GUIDELINE DESCRIPTION

The document content 5 chapters with detailed design examples, case studies and FRP detailing.

Chapter 2
Retrofitting examples have been provided by the companies participating in the working group to qualitatively present the principles of retrofitting with adhesive-bonded composites, FRP localisation and application methods. Case studies are provided in Appendix 2. These documents, which mainly come from product suppliers and installers, do not explicitly address the dimensioning of the reinforcements applied but describe real-life examples of retrofitting which are considered to be representative. They include numerous cases relating to reinforced concrete and prestressed concrete buildings (shopping malls, production facilities, administrative buildings, skyscrapers, etc.), bridges (often old) as well as examples of application to masonry and civil engineering structures (industrial stacks). The retrofitting context is often that of seismic upgrading and/or repairs (even if the case studies proposed are more limited in the second case) that have to be carried out as a result of the repair or reinforcement of structural components to withstand static and/or service loading. In this context, in France, seismic upgrading is simply recommended for bridges while it is compulsory for buildings. In the examples proposed, structural reinforcements are aimed at offsetting inadequate detailing in terms of dynamic loading (overlap or length of rebar anchorage, the quantity of reinforcements required for shear or flexural strengthening, etc.), reinforcing the structure for a new purpose or configuration, or repairing a structure damaged by a previous earthquake. The examples provided show that the reinforcement materials currently applied are often carbon-fibre-based FRPs and that the most frequently reinforced structural component is the column. It is often wrapped with a fabric, generally applied continuously. However, confinement is sometimes completed with flexural strengthening or strengthening designed to offset inadequate detailing (overlap or anchorage lengths). This is achieved by placing FRP strips under the fabric wrapping. Industrial stacks are among the vertical elements that can be retrofitted to enhance flexural strength using FRP strips placed inside or outside the structure. The retrofitting of column/beam joints, as well as horizontal elements in relation to flexure, is also the subject of several case studies. It can be observed that these reinforcements can be anchored to the structure in certain cases. Finally, in several case studies, it appears that FRP retrofitting is not the only solution to be used to strengthen the structure with regards to seismic loading. To meet the requirements of this type of loading, a combination of several retrofitting techniques can be used, including FRP reinforcement. Thus certain structures are reinforced by combining sprayed concrete, additional prestressing, earthquake protection systems and FRPs.

**Chapter 3 : analyse the seismic risk of existing civil engineering works**

This chapter deals with the methods used to analyse the seismic risk of existing civil engineering works. It includes analysis methods and modelling methods using the different approaches defined in paragraph 3.2.3.

Table 1 gives a summary of the methods and approaches used:

<table>
<thead>
<tr>
<th>Modelling methods</th>
<th>Analysis methods</th>
<th>Linear time-history</th>
<th>Pushover</th>
<th>Non-linear time-history</th>
</tr>
</thead>
<tbody>
<tr>
<td>Global approach</td>
<td>Lateral force</td>
<td>Relatively regular structure</td>
<td>Linear time-history</td>
<td>Pushover</td>
</tr>
<tr>
<td>Local approach</td>
<td>Multi-modal</td>
<td>Irregular or complex structure</td>
<td>Relatively regular structure</td>
<td>Ductile design with insulation</td>
</tr>
<tr>
<td>Multi-fibre or</td>
<td>response spectrum</td>
<td>Elastice or ductile design</td>
<td>Ductile design</td>
<td>Insulation</td>
</tr>
<tr>
<td>multi-layer</td>
<td></td>
<td>Elastice design</td>
<td></td>
<td></td>
</tr>
<tr>
<td>approach</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

To validate the retrofitting process, the reinforced structure must be analysed. Effective simulation of the reinforcement is essential. For example, if the strength of the structural member is increased, the composite strips are incorporated into the model as section reinforcements modelled with linear behaviour until failure. They are assigned a high strength and modulus of elasticity corresponding to their real characteristics. The increase in ductility can be induced by the confinement effect on the constitutive law of concrete in compression. The materials are taken into consideration in the models via their static constitutive laws.
Chapter 4: design of FRP

This chapter presents the different FRP reinforcement solutions with respect to the normal force, flexure, confinement and shear force. Since Eurocode 8-3 is entirely devoted to the retrofitting of existing buildings, it is not intended for civil engineering structures. However, in the absence of any other regulations and considering that the behaviour of a bridge pier is similar to that of a building column, the EC8-3 rules will be assumed to be applicable to civil engineering structures in this document.

The field of application of these recommendations covers the design of reinforced concrete components that need to be repaired or reinforced using composite materials (based on carbon, glass and aramid fibres). The design methods presented refer to the Eurocode 8 regulations and lie within their framework. The structural component on which the FRP is to be installed must be sound and free of any pathologies that could reduce the capacity of its surface to lastingly transfer forces to the composite. It is therefore essential to assess the state of repair of the concrete. Given the different reinforcement procedures and in order to ensure a reliable, lasting retrofitting result, it is recommended consulting chapters I, III and IV of the AGFC's guidelines [10] entitled respectively "Recommendations concerning the characterisation of composite materials used to repair concrete structures", "Recommendations concerning in situ implementation of composite materials for reinforcement" and "Recommendations concerning in situ inspection of composite materials for reinforcement", in addition to standards NF P 95101 and NF EN 1504. First, it is important to carry out a global analysis of the structure in accordance with the provisions of Eurocode 8 and the guidelines for assessing existing structures in relation to earthquakes. However, since seismic analysis is too complex a field to be addressed in detail in this document, this chapter will only provide elements relating to the detailing of composites for seismic retrofitting (accidental ULS load conditions). It must be ensured that the structure is verified not only for ULS but also for SLS load conditions and that the construction phasing is taken into account in relation to the SLS.

CONCLUSION

Fibre-reinforced polymers (FRPs) have been used to retrofit reinforced concrete structures since the 1990s in order to compensate for the flexure, shear and compression deficiencies of this type of construction. Now that the effectiveness of these materials has been largely demonstrated, they have become the subject of numerous international regulations and are now widely used in France. Due to the development of regulations aimed at achieving an acceptable seismic performance and ensuring that the designed structure can withstand minor earthquakes without damage, moderate earthquakes with minor non-structural damage and major earthquakes without collapsing, strategies to retrofit existing structures need to be developed. Among these, local retrofitting of the structure can be used to treat a type of component which would probably fail and lead to overall damage. It is also possible to treat the structure as a whole but this would require costly large-scale work that would need to be validated both technically and economically. FRPs not only offer the possibility of improving the overall strength of structural components but also of substantially improving their ductility. Eurocode 8 part 3 describes several design methods for this type of bonded composite reinforcement, mainly with respect to shear. The present document provides a summary of these methods while making suggestions (in blue) to improve and adapt several equations. The aim of this document is to provide in-depth information on the use of FRPs for the seismic retrofitting of existing structures. These guidelines are a complement to existing guidelines on earthquake engineering. In particular, a number of case studies are presented in which FRPs are used for the seismic retrofitting of columns and beams in buildings and civil engineering works.

The working group then describes the specific aspects of the work carried out and the lack of sufficient information on the behaviour of these materials when used for seismic retrofitting. The results are as follows:

- The case studies examined indicate that composite reinforcements are mainly used during seismic retrofitting to compensate for a lack of confinement or shear capacity.

- All retrofitting projects must take into account the state of repair of the structure and the initial construction period in order to define the most appropriate reinforcement method.

- A need for non-linear modelling of reinforced systems would seem necessary to gain a better understanding of the interaction between local and global reactions to seismic loading.
- Little research has been done on the mechanical behaviour of composites under load in the case of flexure with and without axial force in the case of extensive damage to concrete substrates which explains why design methods are only proposed in cases of limited ductility.

- Likewise, it seems essential to consolidate our knowledge of the mechanical behaviour of confined column/beam junctions not only with respect to design and dimensioning but also to modelling in order to incorporate approaches of the push-over type.

- With regard to anchorage, the validation and standardisation of characterisation procedures is also essential.

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Renforcement des ouvrages d'art par matériaux composites


[8] ISIS Canada, Manual No. 4 Strengthening Reinforced Concrete Structures with Externally-Bonded Fibre Reinforced Polymers (FRPs), 2001


ABSTRACT

In many developing countries, existing reinforced concrete (RC) buildings, which were built several decades ago, were constructed with poor quality concrete and insufficient reinforcement details. Experiencing heavy damages after earthquakes, it is clear that seismic retrofit is required to minimize losses. In this paper, the results of an experimental study on seismic performance of pre-damaged and undamaged substandard RC columns retrofitted using aramid fiber reinforced polymer (AFRP) rods are presented. The columns were tested under constant axial and reversed cyclic lateral loads in the column weak-axis direction. Based on the outcomes of the experimental study, it is observed that significant enhancement is possible in flexural strength through the proposed retrofitting method up to 4% lateral drift ratio, which is generally sufficient against seismic actions.

KEYWORDS
Aramid, column, flexural performance, FRP, pre-damage, reinforced concrete, cyclic.

INTRODUCTION

Since most of the existing substandard RC structures, besides other deficiencies, suffer from lack of sufficient flexural strength, they need to be seismically retrofitted to reduce their vulnerabilities against seismic actions. Out of many obstacles for proper seismic retrofitting of these structures, three of them are remarkable: economic constraints, disturbance to the occupants, and disruption of functions of the structure. Traditional retrofitting methods such as concrete and steel jacketing, are not applicable in many cases due to high disturbance to occupants and long duration of retrofitting, which may be very critical for commercial, industrial and public buildings. Since FRPs offer feasible and innovative solutions for seismic retrofitting due to their lightweight, high tensile strength and noncorrosive character, in recent years, construction industry has shown significant interest in the use of FRPs (CEB-FIB, 2001; Lam and Teng 2003; ACI440-2R-08, 2008; Bank 2013). To the best knowledge of the authors’, reversed cyclic flexural behavior of RC members strengthened with longitudinal FRP reinforcement was studied for the first time by Ilki and Kumbasar (2002) and followed by Barros and Dias (2006), Bournas and Triantafillou (2009), Goksu et al. (2012) and Faustino and Chastre (2015) and Seyhan et al. (2015) also studied this type of behavior.

In this paper, the results of an experimental study, which was undertaken to investigate the effectiveness of longitudinal AFRP rods for seismic flexural retrofitting of pre-damaged RC columns, are presented. Aiming at this, two cantilever RC columns were constructed using low-strength concrete and plain reinforcing bars for representing structures, which were designed and constructed without considering seismic resistant design rules. Both specimens were retrofitted with embedded longitudinal AFRP rods to obtain enhancement in flexural strength. One of the retrofitted specimens was damaged before retrofitting. According to the best knowledge of the authors’, there are no comprehensive data which clearly showed the effectiveness of flexural seismic retrofitting of pre-damaged substandard columns using AFRP rods. The tests were executed under reversed cyclic lateral and constant axial loads in the weak-axis direction of the columns. A very limited amount of experimental data exists on seismic behavior of columns in case of FRP retrofitting in the weak-axis direction (Ghatte et al. 2015, Bousias 2004). The efficiency of the proposed retrofitting method is examined considering the indicators of seismic performance such as strength, drift and energy dissipation capacities.

DESCRIPTION OF THE EXPERIMENTAL CAMPAIGN

Specimens
Two symmetrically reinforced cantilever RC columns were constructed (Figure 1a). The specimens were designed to fail in flexure. The cross-section dimensions of the columns were 200 mm×300 mm while the concrete clear cover was 20 mm. The geometric ratio \( A_s/A_c \) of longitudinal reinforcing bars (14 mm) was 1\%, where \( A_s \) is the cross-sectional area of longitudinal steel reinforcing bars and \( A_c \) is the cross-sectional area of the column. The volumetric transverse steel reinforcing bar (10 mm) ratio \( A_{sv} \times D_c/A_c \times \sigma \) of the columns was 0.65\%, where \( A_{sv} \) is the cross-sectional area of transverse steel reinforcing bar, \( D_c \) is the overall periphery of transverse reinforcing bar at a section and \( \sigma \) is the spacing between the transverse reinforcing bars.

One of the specimens (specimen PD) was tested without any retrofit for representing the moderate damaged columns. The specimen PD was loaded to a drift ratio of 2\% and damaged through cyclic loading. It should be noted that at this drift, flexural cracks were occurred along the transverse reinforcing bars and the steel reinforcing bars yielded (Figure 2a). The maximum crack width and the residual lateral displacement were observed to be 1 mm at the column-footing interface and 6.9 mm at the loading height, respectively. This specimen then was retrofitted with embedded AFRP rods in longitudinal direction anchored to the footing. It is worth to note that during retrofitting, the lateral residual displacement was kept as is. Moreover, the specimen was confined with CFRP sheets in transverse direction and nominated as PD-2AR (Figure 2b). The remaining specimen (UD-2AR) was also retrofitted with embedded AFRP rods in longitudinal direction anchored to the footing and CFRP sheets in transverse direction without any pre-damage (Table 1).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Longitudinal FRP reinforcement</th>
<th>Transverse FRP reinforcement</th>
<th>Damage History (before retrofit)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PD</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>PD-2AR</td>
<td>AFRP rods (2x2 (( \phi10 ))</td>
<td>CFRP sheet</td>
<td>Pre-damaged</td>
</tr>
<tr>
<td>UD-2AR</td>
<td>AFRP rods (2x2 (( \phi10 ))</td>
<td>CFRP sheet</td>
<td>Undamaged</td>
</tr>
</tbody>
</table>

Figure 1 (a) Geometry and reinforcement scheme of the specimens, (b) Test setup
Retrofit Application

The specimens UD-2AR and PD-2AR were retrofitted using AFRP rods for enhancing the flexural capacity. Since the damages on the specimen PD were limited, both specimens were retrofitted using the same procedures (without any special measure for the specimen PD-2AR). The retrofitting details and the application steps are presented in Figure 3.

As seen in these figures, firstly the weak concrete cover was removed (Figure 3a). Then, a thin layer of cement based structural repair mortar was applied to obtain a sound and flat surface over the internal steel reinforcing bars (Figures 2b and 3b). As seen from Figure 3b, diagonal scratches were formed on the surface of the cement based structural repair mortar to increase the adhesion to the next layer of mortar. Two AFRP bars of 10 mm diameter were placed symmetrically on each side using an epoxy paste (Figure 3c). Conical holes were opened in the footing for the connection of the AFRP rods to the footing (Figure 2). The 300 mm long part of the AFRP rods were then placed into these conical holes (Figure 2). Subsequently, the conical holes were filled using a high strength epoxy grout. After all, a layer of cement based structural repair mortar was applied over the column surface to bring the specimen cross-section to its original dimensions (200×300 mm) (Figure 3d). Prior to cement based structural repair mortar application, AFRP rods were covered using epoxy primer, which creates a structural bonding bridge between AFRP rods and cement based structural repair mortar. It should be noted that placing the AFRP bars inside the newly formed cement based structural repair mortar, as done in this study and in the studies by Goksu et al. (2012) and Seyhan et al. (2015), is very effective due to prevention of premature buckling of FRP reinforcement under compression. Like many existing substandard RC frame columns, the transverse reinforcement of the specimens was also insufficient causing deficiencies in terms of ductility and shear strength. Therefore, after installation of AFRP rod and formation of concrete cover with cement based structural repair
mortar, CFRP sheets were wrapped around the retrofitted specimens along their height, externally in transverse direction with 200 mm overlap (according to TSDC 2007) at the end of the wrap for enhancing the deformability through confinement and to avoid potential shear failure due to enhanced flexural strength (Figure 3e).

Material Characteristics

The average concrete compressive strength of the test columns was 15 MPa based on the compression tests of 150 mm×300 mm cylinder specimens at the time of testing (after 750 days of age). Concrete was intentionally designed to be of poor quality to represent relatively old existing buildings. All internal reinforcing bars were plain round bars, which have been used commonly until 1990s in Turkey. The average mechanical characteristics of 14 mm diameter longitudinal and 10 mm diameter transverse bars are given in Table 2. In this table, \( f_y, f_{\text{max}}, f_u \) are yield, maximum and ultimate tensile stresses, and \( \varepsilon_y, \varepsilon_{\text{max}} \) and \( \varepsilon_u \) are the tensile strains corresponding to \( f_y, f_{\text{max}} \) and \( f_u \), respectively.

Two different types of FRP reinforcement were used in retrofitting: AFRP rods in longitudinal and CFRP sheets in transverse direction (Figure 4). The geometrical and mechanical characteristics of the aramid and carbon reinforcements are presented in Table 3. In this table, \( t_f, w_f, d_b \) and \( E_f \) are the effective thickness, the effective width, the diameter and the tensile elastic modulus of FRP reinforcement. The compressive strengths of the cement based structural repair mortar used for forming the new concrete cover, the epoxy paste used for bonding the reinforcements to the member surface, the epoxy primer used as bonding bridge, the epoxy adhesive used for bonding CFRP sheets in transverse direction to the member surface, and the epoxy grout used for anchoring aramid reinforcements in the footing were 50, 75, 60, 75 and 80 MPa (TS-EN 196-1 2009). These values were given by the manufacturer and based on the test results of 40×40×160 mm prisms at 7 days of age.

<table>
<thead>
<tr>
<th>Reinforcing bars</th>
<th>( f_y ) (MPa)</th>
<th>( \varepsilon_y )</th>
<th>( f_{\text{max}} ) (MPa)</th>
<th>( \varepsilon_{\text{max}} )</th>
<th>( f_u ) (MPa)</th>
<th>( \varepsilon_u )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \phi 14 )</td>
<td>296</td>
<td>0.0015</td>
<td>398</td>
<td>0.2092</td>
<td>250</td>
<td>0.3066</td>
</tr>
<tr>
<td>( \phi 10 )</td>
<td>315</td>
<td>0.0014</td>
<td>400</td>
<td>0.2170</td>
<td>270</td>
<td>0.3164</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>FRP</th>
<th>( E_f ) (N/mm²)</th>
<th>( d_b ) (mm)</th>
<th>( t_f ) (mm)</th>
<th>( w_f ) (mm)</th>
<th>Ultimate strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>AFRP (rod)</td>
<td>70000</td>
<td>10</td>
<td>-</td>
<td>-</td>
<td>0.024</td>
</tr>
<tr>
<td>CFRP (sheet)</td>
<td>230000</td>
<td>-</td>
<td>0.166</td>
<td>500</td>
<td>0.015</td>
</tr>
</tbody>
</table>

Test Setup

The columns were tested in their weak-axis direction under combined action of reversed cyclic lateral and constant axial loads (Figure 1b). An axial load of 120 kN corresponding to approximately 20% of the axial load capacity of the columns (without considering the contribution of longitudinal steel reinforcing bars) was applied. Target lateral drift ratios were calculated as the ratio of the lateral displacement of the tip of the specimen (at the axis of actuator applying lateral loads) to the specimen height (from bottom of the column to the height of column at the axis of actuator) in pushing and pulling directions (drift ratios ±0.1%, ±0.25%, ±0.5%, ±1%, ±2%, ±3%, ±4%, ±6% and ±8%). It should be noted that generally drifts larger than 3-4% are expected only under severe or very severe earthquakes.

RESULTS AND DISCUSSIONS

The test results are summarized through theoretical and experimental lateral load-displacement curves and their envelopes (Figures 5-6a). The theoretical lateral load-displacement curves were obtained through moment-curvature analysis. In the moment-curvature analysis, steel reinforcing bars were assumed to behave in an elasto–plastic manner with strain hardening. Longitudinal AFRP reinforcement was taken into account as linear elastic
material in tension while the contribution in compression was neglected. The cross-section analysis was performed to be terminated when the AFRP reinforcement reached the rupture strain observed during the tests, which was around 45% of its rupture strain given by the manufacturer. The contributions of cement based structural repair mortar and confinement effect of CFRP transverse reinforcement were also considered during the analysis. After obtaining the moment-curvature relationships, the total top displacements of the columns were determined considering the elastic and inelastic deformations. The details of the theoretical analysis can be found elsewhere (Seyhan 2015). As shown in Figures 5 and 6a, the specimen PD performed in a ductile manner since it was under-reinforced and designed to fail in flexure. The retrofitted specimens (UD-2AR, PD-2AR) exhibited better performance compared to the specimen PD up to the drift ratio of 3%. At this drift, the enhancement in lateral capacities of the specimens UD-2AR and PD-2AR were around 94% and 123%, respectively, with respect to the specimen REF. The sudden remarkable loss of strength of the retrofitted specimens at around drift ratios of 3% was due to the fracture of AFRP rods at the interface of the column and the footing (Figure 7). This clearly showed that the full capacity of the linear elastic AFRP rod was utilized perfectly. The retrofitted specimens behaved similar in terms of strength after the rupture of the AFRP rod.

Energy dissipation capacities of the specimens are presented in Figure 6b. The energy dissipation capacities of the tested specimens are calculated as the areas enclosed by the load-displacement hysteresis loops. As seen from Figure 6b, it is clear that the energy dissipated by the retrofitted specimens is higher than the specimen PD specimens as expected. It can be concluded from Figures 5 and 6 that pre-damaging did not seem to have a negative effect on the seismic behavior of subsequently retrofitted specimen (PD-2AR). This observation is very valuable, which proves the effectiveness of the proposed retrofitting method for the columns with pre-damage. Since the specimens were wrapped with CFRP sheets in transverse direction along their height, the damage was accumulated at the interface of the column and the footing, where the maximum crack width reached approximately 4 millimeters.

![Figure 5 Lateral load-displacement relationships of the specimens](image)

![Figure 6 (a) The envelopes of hysteresis loops, (b) Energy dissipation capacities of the specimens](image)

![Figure 7 Fractured AFRP longitudinal reinforcements (a) UD-2AR, (b) PD-2AR](image)
CONCLUSIONS

In this experimental study, it is intended to develop a method for flexural strengthening of substandard pre-damaged RC columns in their weak-axis direction. For this purpose, two specimens were produced and then tested under constant axial and reversed cyclic lateral loadings. The retrofitted specimens achieved a significant enhancement in flexural strength up to 3-4% drift ratios. A dramatic loss in strength after this drift ratio was observed due to the rupture of AFRP rods at the interface of the column and the footing. Nevertheless, the ultimate drift ratios reached by the retrofitted columns may be deemed as satisfactory considering the potential drift demands that could occur during seismic actions. The outcomes of the experimental study showed that the proposed retrofitting method may provide an attractive solution for enhancing the seismic performance of substandard columns in terms of flexural strength.

ACKNOWLEDGMENTS

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EXPERIMENTAL AND THEORETICAL INVESTIGATION OF THE SEISMIC BEHAVIOUR OF RC BEAM-COLUMN JOINTS

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ABSTRACT

Beam-column joint behaviour plays a crucial role in the seismic performance of framed Reinforced Concrete (RC) structures. Whereas several studies are available in literature about the behaviour of RC joints, a wide consensus design approach is not yet available. Hence further studies on joint behaviour represent an interesting topic through the seismic and structural engineering. Main aim of this paper is to improve the knowledge on the behaviour of RC beam-column joints and rationale Fibre-Reinforced Polymer (FRP) retrofit design according to capacity design. To this purpose, experimental cyclic tests have been carried out on full-scale beam-column joints designed according to different earthquake levels. Numerical simulations, based on an accurate finite element modelling, were performed by means of the TNO DIANA code. The analyses allowed to evaluate the stress distribution in the joint panels as well as to quantify the strains in the reinforcement bars in the beam. The main results of the experimental tests have been discussed through a comparison between experimental and numerical outcomes. Moreover, a simple analytical model is applied to simulate theoretically the seismic performance and the failure modes of beam-column joints in RC structures. The rationale of the simplified model is to identify the strength hierarchy, evaluating the available capacity for any different potential failure mode (namely failure of the cracked joint, bond failure of the bars passing through it, flexural/shear failures of beams or columns). Based on the validated simplified model, retrofit design possibilities with FRP have been discussed according to modern capacity seismic design principles.

KEYWORDS

Beam-column joints, Structural behaviour, Analysis and Design, Experimental tests, Finite elements, Seismic retrofit.

INTRODUCTION

Valuable procedures for the assessment of existing Reinforced Concrete (RC) structures are at present available, since they have been proposed in the Italian and International Building Codes with innovative rules for seismic analysis. Nonetheless, such procedures are not equally focusing on the behaviour of beam-column joints, whose role has been demonstrated to be crucial in the global seismic behaviour of RC framed structures (Masi and Santarsiero 2013).

Beam-column joints should be designed and detailed accurately. Bond and shear mechanisms are common failure modes of beam-column joints during earthquakes and they are brittle in nature. Even under moderate seismic actions the beam-column joints with deficient reinforcement details have shown poor performances. In an RC moment resisting frame beam-column joints are limited regions where large stresses transfer between the framing elements (i.e. columns and beams). Especially in seismic resistant frames the high demand activates the inelastic capacity of members to dissipate seismic energy, but their exploitation could be unfavourably prevented by poorly designed joints, thus exposing an entire structure, even if correctly designed in its other structural components (Manfredi et al. 2008).
In some cases the beams linking into a joint panel are loaded by moments in the same direction, either clockwise or counter clockwise and the bars at the same level are pulled or pushed in equal direction at both sides of the joint panel. If the column is not wide enough or if the strength of concrete in the joint is low, steel bars bond capacity to concrete is inadequate to balance the stress demand (Lignola et al. 2010).

All these concepts have been evaluated experimentally, found theoretically by means of refined FEM numerical analyses and they represent the conceptual basis of the simplified model adopted to identify the strength hierarchy and the capacity related to any potential failure mode. Hence this represents not only an assessment tool, but also a retrofit design tool to change the failure mode and tune the optimum retrofit intervention to expose the structure at hand to the most favourable failure mode preventing those undesirable.

EXPERIMENTAL PROGRAM

A wide experimental program has been performed within the framework of DPC-Reluis research project (Masi et al. 2008) at the Laboratory of Structures of the University of Basilicata (Masi et al. 2012, 2013). The whole experimental program deals with joints equipped with stiff or wide (flat) beams. The present paper is focused on joints with rigid beams, while some results on joints with wide beams can be found in Masi and Santarsiero (2013). The specimens are representative of one-way external joints belonging to the first storey of a four storey RC building with 3.0 m interstorey height. The characteristics of the two specimens discussed herein are as follows:

1. Earthquake Design level: (i) Specimen T1 - Gravity loads only (i.e. no seismic provisions, Figure 1) matching the building codes in the 70’s in Italy (D.M. 30/05/1972), and (ii) Specimen T5 - Seismic loads (i.e. with up-to-date seismic provisions, Figure 2) assuming a medium seismicity zone with a Peak Ground Acceleration (PGA) at the ultimate limit state, $a_g = 0.25g$, according to OPCM 3274 (2003) that is substantially consistent with the current Italian building code (2008) as well as with the Eurocode 8 (2004).

![Figure 1 Specimen T1, without any seismic provision, and its (B) failure mode](image1)

![Figure 2 Specimen T5, with seismic provision, and its (B+J) failure mode](image2)
2. Deformed steel bars (type B450C) with nominal yield strength $f_Y = 450$ MPa and concrete with mean cylindrical strength equal to $f_c = 21.5$ MPa.

3. Same beam cross section, which is 500 mm high, $h$, and 300 mm wide, $b$, and the same column square section with side dimensions, $s$, of 300 mm as it can be seen from Figures 1 and 2.

4. Axial load on the column equal to $N = 0.15 (s^2 f_c)$. 

The full experimental program is reported in Masi et al. (2012). The rationale to analyse, among many others, these two specimens is the opportunity to compare the behaviour of identical joints moving from a gravity load designed joint to a seismically designed joint for medium seismicity zone. This allows also the predictability of the refined numerical and simplified analytical models to be validated.

Three cycles for each drift amplitude were repeated (Figure 3a) and two different failure modes were observed. Specimen T1 showed a maximum strength $F_{max}$ equal to 18.9 kN at a drift $d(F_{max})=0.45\%$. No cracks in the joint panel were detected and starting from a drift equal to 2.5% buckling was observed in the bottom bars of the beam. A purely flexural mechanism involved essentially the beam with wide sub vertical cracks at the beam-column interface and bar rupture due to low-cycles fatigue (B mechanism, Figure 1).

Specimen T5 showed a maximum strength $F_{max}$ equal to 39.8 kN at a drift equal to $d(F_{max})=1.16\%$. In this case a mixed mechanism was observed, in which the flexural damage to the beam was accompanied by diagonal cracking in the joint panel (B+J mechanism, test T5, Figure 2). Starting from a drift equal to 0.75% a sub vertical crack in the beam at the beam-column interface was detected.

This crack grew with drift, while the first diagonal crack appeared in the joint panel at a 2.5% drift and almost the entire joint panel concrete cover spalled exposing reinforcing bars at 5.5% drift.

The shear demand $V_{jhd}$ in the joint panel can be calculated (IBC 2008) through:

$$V_{jhd} = \gamma_{Rd} \cdot A_{s1} \cdot f_{yd} - V_c$$

and $A_{s1}$ is the top reinforcement area of the beam, $V_c$ is the shear force in column, $\gamma_{Rd}$ accounts for over-strength due to steel strain-hardening (not less than 1.2). $A_{s1}$ is smaller in T1 compared to T5 (i.e. 2.26 cm$^2$ vs 6.03 cm$^2$), hence the higher shear demand explains the cracking in the T5 joint panel in spite of the higher transverse reinforcement, causing high values of the tensile principal stress. However, due to its higher strength, the behaviour of seismically designed T5 joint is globally more favourable (Figure 3).

**NUMERICAL SIMULATIONS**

Numerical simulations have been conducted at the University of Naples adopting the Finite Element code TNO DIANA 9.1. Each specimen was modelled implementing a bi-dimensional mesh with more than 1,500 plane stress nine-node elements. Bar reinforcement elements were modelled by means of uniaxial truss elements with axial stiffness only. The mechanical behaviour of the constituent materials is non-linear. Concrete was modelled by means of a smeared approach both for cracking and plasticity aspects. Post elastic behaviour in tension is governed by exponential softening. The Drucker-Prager plasticity criterion is adopted in compression. The horizontal displacement was increased monotonically and the whole time the vertical load was kept constant. The full numerical activity is described in Masi et al. (2013)

Experimental and numerical force-displacement envelope curves are depicted in Figure 3a showing good agreement. The numerical strain values (Figure 3b) of longitudinal steel reinforcement was always lower than 3% in the case of T1 joint, while it was lower than 5% in the case of T5. This finding corroborates that failure of the steel bars in T5 can be attributed to low-cycles fatigue in tension coupled to buckling in load reversals, being strain values too low for a pure tensile failure.

The principal compression stress field shows (Figure 4a) the formation of a concrete strut and T1 presents a wider strut, while in T5 the average stress is almost doubled mainly due to the higher stress transfer in the joint panel due to the larger amount of longitudinal reinforcement anchoring in the joint.

The crack pattern (Figure 4b) occurred regularly in the beams (in a smeared cracking approach), while the joint panel is mainly uncracked at peak load in T1 specimen: additionally, due to the bottom bars anchoring, cracking slightly penetrates inside the joint panel of the seismically designed joint T5. This, coupled to the principal compressive stress field explains the higher damage found in T5.
It is worth noting that the crack pattern is limited to the lower portion of the beam and of the joint panel because the joint was monotonically loaded in numerical simulations.

ANALYTICAL MODELLING

To improve knowledge on the joint behaviour in terms of capacity design and the subsequent strength hierarchy principles, the so called “quadruple flexural resistances model” (Shiohara 2001) was improved and the main unknowns, assumptions and solutions were deeply updated. The novel model (Bossio et al. 2015) considers three bodies divided by diagonal cracks in the joint panel and interacting by means of bending moment and shear (Figure 5).

The equilibrium of internal forces is taken into account, whereas the compatibility not necessarily is. Concrete and steel compression resultants are merged into a single compression resultant force at beam and column ends. In Figure 5 tension/compression forces in reinforcement layers and concrete are $F_i$, while $C$ are contact forces; $\alpha$ is the column over beam length ratio, and altogether they merge into a nonlinear system in 9 unknowns. The column shear is the main parameter to evaluate all other internal force unknowns, hence column shear capacities are provided for each considered failure mode (Bossio et al. 2015), namely: crushing of concrete strut, conventional joint failure due to longitudinal bars yielding/rupture, bond failure of longitudinal bars due to joint panel limited dimensions coupled to high anchoring demand from bars. These failure modes are coupled to beam and column common modes (i.e. flexural and shear capacity, according to classical structural analysis).

Test T1 is analysed showing the entire table of failure modes (see table in Figure 5) and the failure load is the smallest one among the possible failure loads. This test showed the rupture of longitudinal bars in the beam and for this reason the model predictions are not limited to the yielding of longitudinal reinforcement, but are extended to include also the failure of beam bars occurring at an ultimate stress approximately 23% higher than yielding.

Hence the first triggered failure mode is longitudinal bars rupture at $V_c=15.9$ kN, while joint failure due to bar yielding occurs at $V_c=13.59$ kN. Experimental failure load is 18.9 kN, with an underestimation of about 15%. The
joint shear failure with beam longitudinal bars yielding \( V_c=13.59 \text{kN} \) is almost close to the flexural failure of the beam, hence a minor strengthening of this joint panel allows to move from undesired brittle joint shear failure to the desired ductile flexural failure of beams. Analysing the table of failure modes and related loads a remarkable influence of bond capacity on the performance of such systems is observed; worsening of bar bond could impact on the first triggered failure mode up to smooth bars condition. For instance considering lower bond performance (neglecting specific anchoring solutions), reduces the ultimate capacity in terms of column shear, \( V_c \), to about 3 kN. In the same way the test T5 was modelled and a joint failure due to beam tensile reinforcement yielding at \( V_c=43.6 \text{kN} \) instead of 39.8 kN (the experimental outcome) was found, with satisfactorily agreement.

<table>
<thead>
<tr>
<th>Failure mode</th>
<th>( V_c ) [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam flexure</td>
<td>17.75</td>
</tr>
<tr>
<td>Column flexure</td>
<td>56.46</td>
</tr>
<tr>
<td>Beam shear</td>
<td>156.78</td>
</tr>
<tr>
<td>Column shear</td>
<td>145.89</td>
</tr>
<tr>
<td>Crushing of diagonal strut</td>
<td>99.18</td>
</tr>
<tr>
<td>Tensile yielding, beam bars</td>
<td>13.59</td>
</tr>
<tr>
<td>Tensile yielding, upper column bars</td>
<td>86.40</td>
</tr>
<tr>
<td>Tensile yielding, lower column bars</td>
<td>74.70</td>
</tr>
<tr>
<td>Bond (good bond, ( \tau_{\text{max}}=2.5\sqrt{f_c} ))</td>
<td>25.62</td>
</tr>
<tr>
<td>Bond (ribbed bars, ( \tau_{\text{med}}=1.25\sqrt{f_c} ))</td>
<td>13.00</td>
</tr>
<tr>
<td>Bond (smooth bars, ( \tau_{\text{min}}=0.3\sqrt{f_c} ))</td>
<td>3.15</td>
</tr>
</tbody>
</table>

**Figure 5** Novel simplified model scheme and detailed predictions for experimental test, T1

**DESIGN OF FRP RETROFIT**

Based on the validated simplified model, retrofit design possibilities with FRP are discussed according to modern capacity seismic design principles. Force vectors \( F_9 \) and \( F_{10} \) (Figure 5) allow to include the effects of external retrofit in the model.

In the case of test T5, \( F_9 \) allows to include the effect of the six two legs stirrups (at yielding), hence assuming \( F_9=288.3 \text{kN} \), (however in this case longitudinal reinforcement in beam increases too). In existing structures \( F_9 \) and \( F_{10} \) represent, respectively, external strengthening of existing joints along the horizontal and vertical directions. A uni-axial FRP fabric provides a contribution in horizontal or vertical direction, according to fibres direction, conversely a quadri-axial, for instance, FRP fabric provides contributions in both directions (e.g. if each layer of quadri-axial has a thickness of 0.053 mm, combining the overlapping of inclined layers it is equivalent to 0.1279 mm in both horizontal and vertical directions). The total thickness, \( t_f \), multiplied by the FRP stress provides the force contribution by FRP retrofit. FRP (elastic) stress can be obtained, assuming on safe side a maximum FRP contribution, by Young modulus, \( E_f \), multiplied by the maximum strain assumed equal to debonding value (e.g. 0.4% according to Italian guidelines CNR DT200 R1 2013).

In particular, variations about the number and type of FRP layers have been considered. A Carbon CFRP horizontal uni-axial layer (\( E_f=230 \text{ GPa}, t_f=0.167 \text{ mm} \)) in one or three plies configurations (hence \( F_9=76.6 \text{kN} \) or 230.0 kN) or Basalt BFRP horizontal uni-axial layer (\( E_f=89 \text{ GPa}, t_f=0.210 \text{ mm} \)) in one or three plies configurations (hence \( F_9=37.3 \text{kN} \) or 111.8 kN) have been considered.

Obviously first four failure modes (see Figure 5) are independent on FRP retrofit of joint (to improve them it is required to retrofit beam or columns), while joint failure due to bar yielding or due to bar debond could impact on the first triggered failure. In order to understand which failure mode is independent from the retrofit and which is improvable by using it, results are plotted and compared to the reference case, to represent values of column shear (for each failure mode) related to the amount of retrofit (in terms of force \( F_9 \)) in Figure 6.
It is clearly seen that beam flexural failure is not influenced by FRP retrofit (in fact the relevant curve is perfectly flat and horizontal), conversely other joint failures are improved by FRP (in fact the relevant curves increase as FRP retrofit, i.e. $F_9$, increases). One ply of BFRP is barely sufficient to guarantee a desired ductile flexural beam failure, while even one ply of CFRP prevents the yielding of beam bars in the joint. Bond capacity is improved by FRP (reducing the bond demand) and in the case of good bond, flexural beam failure prevents joint bond failure. It is clearly noted that if smooth bars are adopted, hence in lower bond condition, joint failure can be prevented and the failure mode can be pushed to a more desired one only by applying at least a large amount of FRP, e.g. three plies of CFRP allow $V_c$ triggering flexural beam failure to exceed the $V_c$ triggering bond failure mode into joint. As previously stated, four stirrups in the joint (as in the case of the test T5) correspond to an $F_9$ close to 300 kN, however according to Figure 6 specimen failure should be due to ductile beam flexural triggered failure mode (hence $V_c$ close to 18 kN).

CONCLUSIONS

The behaviour of beam-column joints can strongly influence the seismic performance of RC structures. Experimental results showed different performances, in terms of both failure mechanism and shear-drift behaviour. Numerical simulations by means of accurate F.E.M. models have been performed to better understand the experimental results. Failure modes provided by tests have been essentially confirmed by numerical analyses, as the global shear force vs displacement curve and the local cracking pattern were adequately simulated. F.E.M. results provided the theoretical basis of the simplified model allowing not only to better understand the strength hierarchy, but also to calibrate strengthening design. The basic idea is to prevent the undesired failure modes (e.g. brittle shear failure of joint, but also of beams and columns). Once the desired failure mode is selected, then all failure mode mechanisms presenting lower column shear values should be improved. Hence the proposed simplified model represents not only an assessment tool, but also a straightforward criterion to change the failure mode and tune the optimum retrofit intervention for RC structures, e.g. by means of externally bonded FRP, thus leading the structure to the most favourable failure mode while preventing those undesirable.

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A HYBRID FRP-BASED SYSTEM FOR SEISMIC UPGRADING OF R.C.
COLUMNS: EXPERIMENTAL RESULTS AND MICROPLANE BASED
NUMERICAL SIMULATIONS

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ABSTRACT

The paper presents a hybrid technology tailored for the seismic upgrade of reinforced concrete (r.c.) columns. The proposed system combines steel angles with CFRP wrapping, both conceived to increase column ductility as well as axial, shear and flexural capacities. The system will be described as well as the experimental tests that have been carried out for its validation. In order to better understand the contribution of the steel angles, a robust numerical microplane based approach has been adopted for the interpretation of the test results.

KEYWORDS

FRP, RC beams, steel angle, strengthening, shear, compression, flexural.

INTRODUCTION

FRP-based technologies are worldwide applied, having 1) high stiffness and strength to weight ratios 2) corrosion resistance. Their use is moreover encouraged due to the easy handling and installation. Their pioneering application data back to mid 1980’s and thanks to the effort of many researchers are regulated by consolidated recommendation such as the American (USA) ACI 440.2R-08 (2013), the Italian CNR-DT 200 R1 (2013) and the European FIB (2006). Experimental studies have clearly been the base of the FRP development and the present effort is the implementation of database such as those recently implemented for confinement (Nisticò et al, 2014) and shear application (Rousakis \textit{et al}, 2016). In Italy, despite the existing applications, only recently the use of the FRP has been officially accepted as material for upgrading of existing structures, being required a mandatory step for the material qualification. Among the Italian companies that contributed to the technological development of the FRP system, the INTERBAU has been one of the most active, cooperating with many Italian Universities: the result of the many year research was the development of an hybrid system that, combining steel angle with FRP materials, is particularly tailored for the increase of ductility as well as strength of existing reinforced concrete columns. Preliminary experimental tests data back to 2001: the tests regard axial loaded r.c. columns wrapped with CFRP strips arranged either discontinuously or continuously. Further on different arrangements have been tested (Monti and Liotta, 2007) to validate the shear capacity of CFRP reinforced r.c. beams. The successfully validation of the adopted FRP material encouraged other studies concerning different load conditions, such as: 1) pure axial load (Monti and Nisticò, 2008) 2) axial and horizontal loads (Realfonzo and Napoli, 2009; Realfonzo and Napoli, 2012) 3) flexural (Bocciarelli \textit{et al}, 2013) and shear (Bocciarelli \textit{et al}, 2014) reinforced beams. The experimental tests stimulated the need to predict the structural behavior of this technology by means of robust structural models based on an accurate modeling of both concrete and FRP material. Among the possible models, those based on microplane theory (Bazant and Prat, 1988) have been successfully investigated, simulating at first (Gambarelli \textit{et al}, 2014) the behavior of CFRP confined concrete specimens and then (Nisticò \textit{et al}, 2016) the behavior of r.c. columns shear reinforced by means of CFRP strips. The validation of the models further stimulated the use of microplane models for the simulation of the tests (Bocciarelli \textit{et al}, 2014) concerning the INTERBAU hybrid system. In this framework the paper presents the proposed hybrid system, the carried out experimental tests as well as the numerical simulations.

HYBRID SYSTEM DESCRIPTION

The proposed system (Cersosimo and Nisticò, 2006; Monti and Nisticò, 2008) consists of (Fig. 1) 1) four steel angles adhesively applied to the four column corners 2) a CFRP wrapped system that can be either continuous or discontinuous 3) the gap between the CFRP strips and the concrete surface can be filled (Fig. 1d), or not (Fig. 1c) with thixotropic mortar. The steel angles are preformed in order to have a corner radius equal to the column
corner radius that is usually imposed to be equal to 20/30 mm in order to avoid premature FRP collapse. It is well known that 1) the concrete crack evolution strongly influences the FRP stress and strain state and 2) the presence of stress along directions different than the fiber one reduces the FRP capacity, so that 3) the interaction between FRP and concrete surface can be avoided, leaving a gap between the FRP and the concrete surface (Cersosimo and Nisticò, 2012). The CFRP wrapping clearly serves to confine the columns and to provide an additional shear resisting system. The steel angles increase the axial load capacity as well as flexural moment capacity when are properly connected in foundation or on the beam-column joints in case of frame-structure. Regarding the connection two different solutions can be adopted consisting in a) steel plates welded to the angle, conceived to transmit only tensile force avoiding bar buckling (Fig. 1g) b) a L-shape profile welded to the longitudinal angles and placed in contact with the concrete surface in order to transfer both compression and tension actions.

![Figure 1 Conventional and hybrid FRP wrapping without and with mortar (gray)](image)

The adopted steel angles have a characteristic yield stress of 260 MPa, corresponding to a yield strain of 0.13%: 1) the steel angles side length ranges between 40 and 80 mm and the thickness ranges between 4 and 8 mm. The adopted wrapping system is the CARBOSTRU® C-SYSTEM that consists of a unidirectional High Modulus (390 GPa) CFRP (400 g/m²) combined with an epoxy resin: the equivalent thickness, for each layer, is 0.225 mm. The mechanical macroscopic properties (tensile strength and Young modulus), are usually evaluated testing a 3 layer specimens with fibres oriented in the loading direction: typical obtained values together with the geometrical properties of each specimens, are reported in Table 1 where: t_i and b_i are the average thickness and width, respectively, F_u is the attained ultimate load, E_u and f_u are the Young modulus and the ultimate strength referred to the fibre area. It is well known that the FRP mechanical properties depend on the angle (θ) between the fibre orientations and the applied load, so that they have been evaluated (Nisticò et al, 2016) and reported in Table 2 that includes: 1) specimen dimensions (t_i,b_i in average), 2) average values of the ultimate load (F_u) 3) average values of mechanical properties (f_u,E_u), together with (in parenthesis) their values normalized with respect of the 0° case values. The adhesive properties (shear strength) of CFRP are measured by means of displacement control tests carried out on single-lap shear specimens that consist of two aluminium sheets, overlapped of 60 mm and glued together using the adhesive to be tested which generally has a Young modulus of 12800 MPa: typical shear strength values are close to 6 MPa.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>t_i (mm)</th>
<th>b_i (mm)</th>
<th>F_u (N)</th>
<th>E_u (MPa)</th>
<th>f_u (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.48</td>
<td>20.17</td>
<td>23,126</td>
<td>395,645</td>
<td>1820</td>
</tr>
<tr>
<td>2</td>
<td>2.46</td>
<td>19.78</td>
<td>43,084</td>
<td>395,720</td>
<td>3457</td>
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<tr>
<td>3</td>
<td>2.46</td>
<td>20.004</td>
<td>41,163</td>
<td>386,649</td>
<td>3196</td>
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</table>

<table>
<thead>
<tr>
<th>Orientation</th>
<th>t_i (mm)</th>
<th>b_i (mm)</th>
<th>F_u (N)</th>
<th>E_u (MPa)</th>
<th>f_u (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0°</td>
<td>3.0</td>
<td>20.6</td>
<td>43,600</td>
<td>393,131</td>
<td>1,335 (1)</td>
</tr>
<tr>
<td>10°</td>
<td>2.6</td>
<td>19.9</td>
<td>13,100</td>
<td>148,138 (0.37)</td>
<td>975 (0.3)</td>
</tr>
<tr>
<td>20°</td>
<td>2.9</td>
<td>19.4</td>
<td>5,700</td>
<td>66,943 (0.17)</td>
<td>433 (0.14)</td>
</tr>
<tr>
<td>40°</td>
<td>3.3</td>
<td>20.4</td>
<td>2,000</td>
<td>25,505 (0.05)</td>
<td>149 (0.05)</td>
</tr>
</tbody>
</table>

**EXPERIMENTAL TESTS**

The carried out tests regard 1) concrete columns subjected to axial load (Monti and Nisticò, 2008) 2) r.c. columns designed and tested to emphasize either a) their shear capacity (Bocciarelli et al, 20014) or b) their flexural capacity (Realfonzo and Napoli, 2009/2012).
Confined concrete columns

Compression tests were performed on nine specimens of square (200×200 mm) and rectangular (200×300 mm; 200×400 mm) concrete columns (1400 mm long). All the specimens had rounded corners with a radius of 20 mm. For each typical section, three different configurations were considered: unwrapped (UW), fully wrapped (W) and fully wrapped, with L-shaped steel angles (4×40 mm) placed at the corners (WL). Some results, in terms of Force-Displacement curves are reported in the following Fig. 2, from which can be noticed that the ultimate load increases 1) passing from the un-wrapped (UW) to the wrapped (W) columns and 2) passing from the wrapped (W) to the wrapped column with steel angles (WL), corresponding to which the load increment (~380 kN in average) is almost equal to the yielding force (400 kN) of the four steel angles, meaning that their contribution is fully exploited.

Shear reinforced concrete columns

In the framework of the experimental investigations 10 specimens were tested under three-point bending load. Most of them were 3.45 m in length with their rectangular cross-section reinforced with 6 CFRP intermediate single layer strips and 2 double layers reinforcement at the supports and at the mid-span, were the load is applied. The Hybrid System (denoted as CWL) includes two possibilities having or not the mortar in between the steel angles (40×6 mm); the latter case is denoted with the symbol "*". Concerning the maximum achieved load it has to be observed (Table 3) that 1) the WL configuration has a load increases corresponding to the 17% of the CW configuration without steel angles 2) the CWL* configuration (without mortar) has a load increases corresponding to the 12% of the CWL configuration (with mortar).

| Table 3 Tested specimens: maximum achieved load (N) and example of crack pattern |
|-----------------|-------|------|------|------|
| ID | REF | CW | CWL | CWL* |
| F_{max} | 162,700 | 400,590 | 468,300 | 523,890 |
| CWL | | | |

| 3\textsuperscript{rd} crack | 2\textsuperscript{nd} crack | 1\textsuperscript{st} crack |

Flexural reinforced concrete columns

In (Realfonzo and Napoli, 2009) the results of cyclic and monotonic flexural experimental tests have been presented (see Figure 3 and Table 4): among the others, the tests concern r.c. square (300×300 mm) concrete columns (2200 mm long) reinforced either with smooth or deformed steel bars and CFRP wrapped including or not steel angles (80×6 mm) that can be connected in foundation by means of the devises reported in Fig. 1 (f,g,h). Two values of the normalized axial load (ν) have been considered which correspond to the 14% and 40% of the ultimate load. The test highlighted that 1) the placement of longitudinal steel angles not connected to the foundation give higher ductility 2) the placement of longitudinal steel angles connected to the foundation considerably increases the flexural strength being however characterized by a reduction of the available ductility. The experimental campaign has been extended (Realfonzo and Napoli, 2012) including rectangular sections: results from cyclic tests performed on full scale RC columns (300×700 mm) allowed to conclude that despite the high side ratio, considerable ductility increases is achieved and that the proposed hybrid system works well in terms of flexural strength even if loss of ductility are consequent to the brittle failure of the connections. General remarks are that 1) the hybrid system improves the column performance in terms of ductility 2) improvements (under development) in the design of the connection are needed if strength increase has to be combined with ductility increase.
Table 4 Steel and concrete mechanical properties for Columns (C) with Smooth (S) and Deformed (D) rebars

<table>
<thead>
<tr>
<th></th>
<th>C</th>
<th>f_{cy} (MPa)</th>
<th>ε_{u} (%)</th>
<th>f_{cm} (MPa)</th>
<th>f_{cm} (MPa)</th>
<th>f_{cm} (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S</td>
<td>346</td>
<td>0.165</td>
<td>498</td>
<td>23.8 (C13-S-C)</td>
<td>28.9 (C3-S)</td>
<td>35.3 (C11-S-A1)</td>
</tr>
<tr>
<td>D</td>
<td>556</td>
<td>0.265</td>
<td>655</td>
<td>16.73 (C9-D)</td>
<td>31.8 (C9-D)</td>
<td>22.1 (C7-D-C)</td>
</tr>
</tbody>
</table>

Figure 3 Steel reinforcement and Force–displacement relationships for cyclic loads: smooth rebars (left) and deformed rebars (right). 1) Black = Unreinforced  2) Red = Carbon Wrapped  3) Green = Carbon Wrapped with steel angles connected in foundation with the device reported in Fig. 1g (specimen C11-S-A1) and Fig 1h (specimen C15-D-A1)

MICROPLANE BASED NUMERICAL SIMULATIONS

The previous described system represents a reliable system for RC column upgrade: the experimental tests confirmed its validity for some load conditions and further investigations are needed to evaluate the system capacity for others, such as the biaxial flexural condition, interacting with shear and axial load, that represents “the condition” for seismic action. In this direction the support of numerical models are for sure to be considerate before going forward with further experimental investigations: the selected model is based on microplane theory (Bazant and Pratt, 1988) that has been successfully validated for FRP confined specimens (Gambarelli et al, 2014) and for FRP shear reinforcement (Nisticò et al, 2016) having reproduced the experimental results of the shear reinforced column (CW) presented in the previous section: the analyses have been carried out with the program MASA (Ozbolt, 1988). Regarding shear reinforcement, the reproduced crack pattern (Figure 4) is in agreement with the experimental one (see figure in Table 3): 1) microplane models, have been adopted for both concrete and FRP, considering material non linearity 2) steel bars have been modelled by means of non linear frame elements 3) the contact between FRP and concrete has been modelled with contact elements which include shear as well as axial non linearity. It is worth noticing that the feature of the adopted model (Ozbolt et al, 2001) for FRP material was fundamental for the success of the carried out analyses, and to correlate the CFRP stress evolution (Figure 4) with the crack opening: 1) the strips are not uniformly loaded 2) the maximum stress reached in the FRP is approximately equal to 2000 MPa, which corresponds to approximately 70% (see Table 2) of the average experimental strength value when the fibre orientation is parallel to that applied load. 3) the reduction of 30% in strength can be attributed to the direction of maximum principal strains which form, with the fibre direction, an angle of approximately 8°.

Figure 4 CW column: 1) crack pattern and Force (kN) – Displacement (mm) curves (left) 2) FRP stress (MPa) distribution (centre and right)
The microplane based approach has been also applied (Polimanti, 2013) to simulate one column reinforced with the hybrid system when the mortar between the steel angle is not considered (CWL* case). The model are the same of the previous cases, specifying that: 1) the steel angle have been modelled with solid elastic elements having the steel material properties 2) the contact between steel angle and concrete as well as FRP, has been modelled through the non linear contact element previously introduced.

In the following Figure 5, the reproduced crack pattern, at the first diagonal crack propagation, is reported together with the stress evolution in the FRP sheets along the beam height (H*): it can be observed, comparing it with the one obtained without steel angle, that: 1) the FRP stress is more distributed among the strips 2) the maximum reached stress (≈ 1800) corresponds to approximately 65% of the average experimental strength value when the fibre orientation is parallel to that applied load 3) even if a gap exists, in between the concrete and FRP, there is an increases, with respect to the WL case, of the angle between the maximum principal strains and the fibre direction.

The crack pattern reported in Figure 6, outlines 1) the activation of two diagonal cracks (6a,d) which are present in the experimental test (see Figure in Table 4) and 2) the evolution of the strips activation that starts from the closest to the load (#1) and then involve in sequence the strips #2,3 2) the role of the localization which consequence is the not symmetric structural behavior characterized by a suddenly collapse of the left side strips after the propagation of the second crack. The proposed model can be considered satisfactory in reproducing the experimental test of CW columns, however more studies, currently in progress, have to be done in order to justify the experimental evidences of the proposed hybrid system.

![Figure 5 CW* column: 1) crack pattern and Force (kN) - Displacement (mm) curves (left) 2) FRP stress (MPa) distribution (centre and right)](image)

![Figure 6 CW* column: crack propagation](image)

![Figure 7 CW (left) and CW* (right) column: FRP principle strain angle](image)
CONCLUSIONS

The paper presented an hybrid system conceived to increase the seismic capacity of r.c. existing columns. The system, consisting in a combination of steel angle and FRP strips, have been validated through experimental tests performed to evaluate the axial as well as the shear and flexural capacities. The carried out tests were the base for microplane based models that should be more investigated to capture the complex phenomena that characterize the strong interaction between 1) concrete 2) steel angle and 3) FRP strips of the proposed hybrid system. At the present can be deduced that the use of steel angle, in the hybrid system, has a double effect that combines the shear strength increase due to the steel angles and the CFRPs characterized by better distributed stress among the strips.

REFERENCES

ABSTRACT

The Japanese construction community started to use fiber reinforced polymers in mid-1980’s. Initially the FRP rod or strand was used as a substitute of steel reinforcing bars to avoid corrosion of reinforcement or invasion of electric stray current. An application of FRP sheet started for seismic retrofit of buildings in late 1980’s. The 1995 Kobe Earthquake and the 1999 Design Guidelines of FRP on Seismic Retrofit of RC Buildings accelerated the use of FRP sheet for seismic retrofit of buildings in damaged area since light-weight of FRP sheet makes it easy to ship and construct. Target members for FRP sheet retrofit are normally line elements such as beams and columns. Although RC walls may not be the best target to strengthen with FRP sheet, it increases the shear capacity with relatively easy construction. FRP sheet is not only used for increasing shear capacity of RC walls, but it can be also used to increase impact resistance of tsunami debris. This paper introduces two cases of FRP sheet application to RC walls. The first application is the use of soft fiber reinforced polymers to increase the shear resistance of RC walls. The second application is the use of CFRP to increase the impact resistance of RC walls against tsunami debris. Experimental programs on two cases show the effectiveness of FRP sheet to resist against in-plane shear force and out-of-plane impact force.

KEYWORDS

FRP, RC walls, shear strengthening, impact loading, tsunami debris.

INTRODUCTION

Table 1 shows a history of major seismic codes and standards with related events. The civil engineering society started to study the application of fiber reinforced polymers (FRP) to reinforced concrete structures in 1980’s in Japan. Shinmiya Bridge in Hakui city is the first concrete bridge prestressed with carbon fibre reinforced polymer strands to avoid salt damage. An application of FRP sheet started for seismic retrofit of buildings in late 1980’s. Carbon FRP sheet was used for the first time to retrofit a reinforced concrete chimney in 1994. FRP was considered a good construction material due to its light-weight, good durability, and proper stiffness and strength but it was not widely used due to its high cost. The 1995 Kobe Earthquake and the Design Guidelines of FRP on Seismic Retrofit of RC Buildings published by the Japan Building Disaster Prevention Association (JBDPA 1999) accelerated the use of FRP sheet for seismic retrofit of buildings in damaged area since light-weight of FRP sheet makes it easy to ship and construct. Qualification programs by the Fibre Repair and Strengthening (FiRSt 2016) established in 1999 also enhanced the usage of FRP sheet since seismic retrofit of reinforced concrete with FRP sheet necessitates meticulous attention to details for better seismic behaviours. This paper introduces two unique cases of FRP sheet application to RC walls.

CASE 1: INPLANE STATIC EXPERIMENT ON WALLS WITH SRF

FRP with low Young’s modulus (SoftFRP) was developed in late 1990’s and early 2000’s. They were manufactured from relatively economical materials like polyacetar (Iihoshi 2005) or polyester (Igarashi 2000) to substitute expensive carbon fiber reinforced polymers (CFRP) in the beginning. FRP with high Young’s modulus like CFRP may fracture when it experiences large tensile strain at cracks. However, it was found that a large deformation capability of SoftFRP was effective to strengthen concrete structures since it is highly resistant to the local concentration of tensile strain at cracks.
Super Reinforcement with Flexibility (SRF) was developed as a construction material in 1999 and considered as one of SoftFRP’s. Its Young’s modulus is 1/40 of that of CFRP or ordinary mild steel. The tensile strength is 1/35 of CFRP and similar to that of ordinary mild steel. SRF costs 2/3 of CFRP and is easier to purchase locally. The research on SRF retrofit of RC columns was originally published in 2000 (Igarashi 2000). Since then, retrofit schemes on RC columns have been studied (Kabeyasawa et al. 2010). SRF obtained good reputation for retrofitting columns and beams of buildings and bridges. Studies on SRF retrofit of RC walls began in mid-2000’s (Sanada et al. 2005, Kabeyasawa et al. 2007, Warashina et al. 2008). This paper shows performance of RC walls strengthened with SRF by introducing an experimental work (Kono et al. 2008).

<table>
<thead>
<tr>
<th>Important event</th>
<th>Building Standard Law</th>
<th>Seismic retrofit promotion act</th>
<th>Actions taken by the Japan Building Disaster Prevention Association (JBDPA) and the Architectural Institute of Japan (AIJ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1968: Off the Coast of Tokachi EQ (Shear failure of RC columns)</td>
<td>1971: Minor revision of Building Standard Law (to prevent shear failure)</td>
<td>1997: Seismic evaluation and retrofit for existing RC buildings (JBDPA)</td>
<td>1995: Seismic Retrofit Promotion Act</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 2 Mechanical Properties of SRF Sheet</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width (mm)</td>
</tr>
<tr>
<td>------------</td>
</tr>
<tr>
<td>100</td>
</tr>
</tbody>
</table>

Experimental setup

Two identical structural wall specimens were constructed with eccentric openings as shown in Figure 1(a). L1 is a prototype without any strengthening and abide by the 1999 AIJ standard. L4 had identical configuration to L1 but was strengthened with SRF sheet. A single layer of SRF sheet was applied on the south face of L4 and anchored to the north face as shown in Figure 1(b). The beams of the second and third floor (B2 and B3) next to the openings...
were also strengthened with a single layer of SRF. Mechanical properties of SRF sheet is listed in Table 2. Polyurethane adhesive was used for placing SFR on the concrete surface.

The lateral load was applied to the loading beam as shown in Figure 1(c) and the drift at the midspan of loading beam was controlled. The loading protocol was two cycles each at drift angles of 0.05%, 0.1%, 0.25%, 0.5%, 0.75%, 1.0% and 1.5%. The vertical load was applied to the top of each column so that the contraflexure point stayed 2500 mm above the foundation beam, that is, shear span to depth ratio was 1.0.

Experimental Results

Lateral load – drift angle relations are shown in Figure 2. It is clear from L4 that SRF increased the shear capacity without changing the initial stiffness. The degradation of the shear strength from the positive peak to R=+1.3% in L4 was gentler than that of L1.

When the shear strength of L4 was computed, the effect of SRF was taken into account by adding the equivalent amount of shear reinforcement steel to the equation. The equivalent amount of steel was computed so that the steel at yielding carries the same magnitude of force which a given amount of SRF carries at the strain of 0.57%. The computed and experimental shear strengths agree well in the positive direction and the employed equation is considered reasonable. The first shear crack of L1 was found at the first floor wall panel at drift angle (R) of +0.04%, and the number of cracks kept increasing till R=0.5%. Then the vertical reinforcement buckled at the wall base of the first floor at the region enclosed by a circle. The concrete started to spall locally from this drift angle. Then the shear sliding failure took place along the wall base at R=+1.5% and the lateral load dropped suddenly. L4, strengthened with SRF sheet, had the initial shear cracks at R=+0.05% and the number of cracks increased until R=0.5%. At the part of the second floor beam (B2), flexural cracks opened widely and the maximum load was reached at R=0.68% in the positive direction. Peak load in negative direction was reached when the shear failure occurred at the third floor wall panel since the force path was not secured due to an opening. In positive direction, the wall resisted the external force until R=1.25%. At R=1.25%, degradation of load carrying capacity since the sliding shear failure occurred just below the loading beam.

Shear strength of the structural wall (L4) in the positive direction was enhanced by 27%. Both SRF and polyurethane adhesive have low stiffness and high deformation capability. With SRF attached on concrete surface, cracks did not concentrate locally but rather distributed evenly over wall panels, and consequently the number of cracks increased and the width of each crack decreased. The cracks with smaller width made the aggregate interlock more effective resulting in the enhancement of shear capacity. SRF also alleviated deterioration of the load carrying capacity by preventing the spalling of concrete. Even after the concrete strut failed in compression, damaged concrete stayed in the original position since SRF held it. Consequently, the degradation after the peak was gentler. Unfortunately, the ductility enhancement due to SRF was not clear due to an unexpected failure mode.

This chapter introduced SRF, one of fiber reinforced polymers with low Young’s modulus. It was used to retrofit RC structural walls with openings. Its large deformation capability improved seismic performance of structural walls and relieved stress concentration. In addition to the force resisting mechanisms stated here, SRF has a function to relieve stress concentration due to its large deformation capability. Sharp corners of the RC members do not have to be round like they should be for CFRP sheet which is sensitive to sharp corners. SRF can be also applied to large bridge columns, brick-walls and lumber structures as well for its design flexibility, ease of handling at construction and low price.
CASE 2: OUT-OF-PLANE IMPACT RESISTANCE OF WALLS WITH CFRP

The Japanese current building codes and standards for RC buildings do not have any statements on design against tsunami debris impact loads. Since there are not many experimental studies on local damage of RC buildings by tsunami debris impact load, experimental data is necessary to develop design methods. This paper presents a test result of RC walls strengthened with FRP sheet to see its effectiveness on the impact resistance (Watanabe et al. 2016).

Experimental setup

Figure 3(a) shows configuration and dimension of square wall specimens and (b) shows the layout of FRP sheet. Table 3 shows test variables with observed failure modes. The specimens are 50% scaled wall models with 1300mm by 1300mm in planar size and 120mm in thickness. Design concrete strength was 24 N/mm². Series STD were ordinary reinforced concrete walls with minimum reinforcement required by the 2010 AIJ standard and without further strengthening. Series PUA and PUT had polyurea and polyurethane resin polymer membrane of 2mm thickness, respectively. Series FRP had CFRP sheet in two transverse directions as shown in Figure 3(b). Series FPA had 2 mm thick polyurea resin polymer on top of CFRP sheet placed in two transverse directions. All strengthening was conducted only on the rear face of the specimens while impactor was designed to hit the front face. Their performance target was to prevent penetration of impactor and shattering of rear side concrete. Each series consisted of four identical specimens which were tested under different impact velocity ranging 6.54m/s ~ 13.25m/s and impactor weight, either of 104.95kg or 152.25kg as listed in Table 3. Mechanical properties of materials are listed in Table 4.

Figure 4 shows typical failure modes for each series. Red lines represent cracks for Series STD and represent boundary of delamination of strengthening materials in other series. The shading in Figure 4 (b-1) represent area of concrete spalling. Specimen STD-c, which had the impact velocity of 8.07 m/s, had a mix failure mode of scabbing and perforation. Specimen STD-d (Figure 4(b-1)) (9.07 m/s) had perforation and steel reinforcement relieved flying of damaged concrete fragments. For Series PUA and PUT, epoxy resin was torn (Figure 4(a-2))

### Table 3 Specimens and test variables

<table>
<thead>
<tr>
<th>Name of Specimen</th>
<th>Reinforcement (reinforcement index in %)</th>
<th>Compressive strength of concrete (N/mm²)</th>
<th>Weight (kg)</th>
<th>Velocity (m/s)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>STD-a</td>
<td>CD5# double 100 double (0.33%)</td>
<td>19.9</td>
<td>104.95</td>
<td>6.42 Scabbing</td>
<td></td>
</tr>
<tr>
<td>STD-b</td>
<td></td>
<td></td>
<td>7.39</td>
<td>Scabbing</td>
<td></td>
</tr>
<tr>
<td>STD-c</td>
<td></td>
<td></td>
<td>8.07</td>
<td>Scabbing &amp; perforation</td>
<td></td>
</tr>
<tr>
<td>STD-d</td>
<td></td>
<td></td>
<td>9.07</td>
<td>Perforation</td>
<td></td>
</tr>
<tr>
<td>PUA-a</td>
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<td>Tearing</td>
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<tr>
<td>PUA-b</td>
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<td>PUA-1</td>
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<td>9.18</td>
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<td>FRP-a</td>
<td>CD5# double 100 (0.33%)</td>
<td>25.6</td>
<td>104.95</td>
<td>10.34 Delamination</td>
<td></td>
</tr>
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<td>FRP-b</td>
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<td>152.25</td>
<td>8.67 Delamination</td>
<td></td>
</tr>
<tr>
<td>FRP-1</td>
<td></td>
<td></td>
<td></td>
<td>10.42 Perforation</td>
<td></td>
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<tr>
<td>FRP-2</td>
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<td></td>
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<td>12.11 Delamination</td>
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<tr>
<td>FPA-a</td>
<td></td>
<td></td>
<td>104.95</td>
<td>12.80 Delamination</td>
<td></td>
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<tr>
<td>FPA-b</td>
<td></td>
<td></td>
<td></td>
<td>12.41 Delamination</td>
<td></td>
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<tr>
<td>FPA-1</td>
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<td></td>
<td>152.25</td>
<td>8.32 Delamination</td>
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<tr>
<td>FPA-2</td>
<td></td>
<td></td>
<td></td>
<td>10.5 Perforation</td>
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<tr>
<td>PUT-a</td>
<td></td>
<td></td>
<td>25.8</td>
<td>6.54 No damage</td>
<td></td>
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<tr>
<td>PUT-b</td>
<td></td>
<td></td>
<td>23.2</td>
<td>7.55 No damage</td>
<td></td>
</tr>
<tr>
<td>PUT-c</td>
<td></td>
<td></td>
<td></td>
<td>10.50 Tearing</td>
<td></td>
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<tr>
<td>PUT-d</td>
<td></td>
<td></td>
<td>25.8</td>
<td>13.08 Perforation</td>
<td></td>
</tr>
</tbody>
</table>

*1: Velocity was measured with a hi-speed video camera except specimens with *2 whose velocity was measured with a laser beam sensor

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and (a-3)) when the impact velocity was close to 10 m/s, perforation happened when the velocity became as fast as 13 m/s (Figure 4(b-2) and (b-3)). In Figure 4(a-2) and (a-3), polyurea and polyurethane membrane stretched due to the scabbing of concrete but damaged concrete fragments stayed inside polyurea or polyurethane membrane. For Series FRP and FPA, perforation did not happen even if the velocity became 13 m/s with a 104.95 kg impactor. Hence, the impactor weight was increased to 152.25 kg and the impact test was continued. Then perforation was observed for velocity of 10.45 m/s as shown in Figure 4(b-4) and (b-5).

Table 4 Mechanical properties of materials

<table>
<thead>
<tr>
<th>Type</th>
<th>Yield Strength (MPa)</th>
<th>Yield strain</th>
<th>Young's modulus (MPa)</th>
<th>Tensile strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CD5</td>
<td>522.0</td>
<td>0.448</td>
<td>2.18 × 10^5</td>
<td>586.7</td>
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Table 5 Limit velocity and limit thickness for perforation

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Compressive strength of concrete (MPa)</th>
<th>Young's modulus (MPa)</th>
<th>Splitting tensile strength (MPa)</th>
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</thead>
<tbody>
<tr>
<td>STD, PUA</td>
<td>19.9</td>
<td>2.02 × 10^4</td>
<td>1.76</td>
</tr>
<tr>
<td>PUT-b, -c</td>
<td>23.2</td>
<td>1.95 × 10^4</td>
<td>1.91</td>
</tr>
<tr>
<td>FRP, FPA</td>
<td>25.6</td>
<td>2.19 × 10^4</td>
<td>2.08</td>
</tr>
<tr>
<td>PUT-a, -d</td>
<td>25.8</td>
<td>2.15 × 10^4</td>
<td>2.02</td>
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</tbody>
</table>

Table 6 Mechanical properties of composites

<table>
<thead>
<tr>
<th>Type</th>
<th>Thickness (mm)</th>
<th>Tensile strength (MPa)</th>
<th>Fracture strain (%)</th>
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<tr>
<td>Polyurea resin composite</td>
<td>2.0</td>
<td>25.5</td>
<td>222</td>
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Table 5 Limit velocity and limit thickness for perforation

<table>
<thead>
<tr>
<th>Fastest velocity after specimen</th>
<th>V_{exp} (m/s)</th>
<th>Mass (kg)</th>
<th>Concrete splitting tensile strength (MPa)</th>
<th>Proposed method</th>
</tr>
</thead>
<tbody>
<tr>
<td>PUA-a</td>
<td>10.25</td>
<td>104.95</td>
<td>1.91</td>
<td>V_{exp}/V_c = 1.14, e = 0.133, t/e = 0.90</td>
</tr>
<tr>
<td>PUT-c</td>
<td>10.50</td>
<td>152.25</td>
<td>2.08</td>
<td>V_{exp}/V_c = 1.16, e = 0.136, t/e = 0.88</td>
</tr>
<tr>
<td>FRP-1</td>
<td>8.67</td>
<td>104.95</td>
<td>9.02</td>
<td>V_{exp}/V_c = 1.11, e = 0.131, t/e = 0.92</td>
</tr>
<tr>
<td>FPA-1</td>
<td>8.32</td>
<td>152.25</td>
<td>7.81</td>
<td>V_{exp}/V_c = 1.07, e = 0.127, t/e = 0.95</td>
</tr>
</tbody>
</table>

Nakamura et al. (2015) proposed the design equation for scabbing and perforation limits by modifying Hughes’ equation (Hughes 1984). If the ratio of experimental limit velocity (V_{exp}) to computed limit velocity (V_c) is larger, the velocity of perforation increased from that for a plain reinforced concrete wall due to strengthening. Table 5 shows that V_{exp}/V_c ranged from 1.07 to 1.16, the strengthened series have 7 - 16% increase of limit velocity for perforation compared to plain reinforced concrete walls. The strengthened series decreased the limit thickness of perforation to 88 – 95% of the plain reinforced concrete for an arbitrary impactor velocity.

CONCLUSIONS

Reinforced concrete walls strengthened with SRF

Fiber reinforced polymers with low Young’s modulus was used to increase the shear capacity of an RC structural wall specimen. The large deformation capability of SRF improved shear capacity of structural walls by 27% and 9% in the positive and negative directions, respectively, and relieved damage due to excessive stress concentration.
Reinforced concrete walls strengthened with CFRP
Four type of strengthening method (CFRP sheet, polyurea and polyurethane resin polymers, and combination of CFRP sheet and polyurea resin polymers) increased the velocity of penetration, scabbing and perforation and relieved flying of shattered concrete fragments behind the impact face.

- As impact velocity increased, failure modes changed from delamination of CFRP sheet to perforation for wall specimens strengthened with CFRP polymer sheet and with or without polyurea resin composite. On the other hand, failure modes changed from tearing to perforation for wall specimens with polyuria resin composite or with polyurethane.
- With strengthening materials used in this test, the limit velocity of perforation increased by 7% - 16% and the limit thickness for perforation can be decreased by 5% - 12% as conservative assessment.

ACKNOWLEDGMENTS

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RELOCATING THE PLASTIC HINGE AT BEAM COLUMN JOINT INTERFACE INTO THE BEAM IN CFRP STRENGTHENING

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ABSTRACT

Reinforced concrete buildings with moment resisting frames and beam-column joints (BCJ), designed prior to introduction of seismic codes, are deficient when subjected to lateral seismic loading. The weakest link in these buildings, the BCJ fail in shear under lateral seismic loads resulting in irreparable damage to the buildings and possibly a collapse. Several strategies for shear strengthening deficient joints in old buildings have been investigated and implemented in buildings over the past three decades using CFRP sheets and laminates. The shear failure in joints is successfully precluded, with the failure mode shifting to the formation of a plastic hinge at the BCJ interface. A localized failure in the connecting beams away from the joint is a more preferable failure mode rather than a plastic hinge formation at the interface. In modern BCJ with high strength concrete and shear reinforcement in joints, the plastic hinge is also formed at the BCJ interface, which is again undesirable failure mode. It is highly appropriate that all BCJ should undergo a flexural-shear failure with the plastic hinge formation away from the BCJ interface well into in the connecting beam. Based on a nonlinear finite element investigation of CFRP strengthened shear-deficient BCJ using concrete damage plasticity model in ABAQUS, experimental investigations have been conducted. The experimental investigation shows that in BCJ strengthened in shear, an appropriate scheme of CFRP strengthening results in improved ductility, energy dissipation and an increase in load capacity with a movement of the plastic hinge formation from the BCJ interface to a distance inside the beam.

KEYWORDS

Beam column joints, Plastic hinge, CFRP strengthening, ABAQUS.

INTRODUCTION

Reinforced concrete buildings are designed to withstand the moment induced in structural members from gravity, wind and earthquake loadings. Moment-resisting frames comprising beams, columns, and beam-column joints are the main elements resisting these loadings. A beam-column joint (BCJ) is that part of the RC-column, which lies within the depth of the deepest beam that frames into the column and is the most critical region which is subjected to large forces in a severe seismic event (Moehle 2014). The behavior of the BCJ has a great influence on ductility, energy dissipating capability and transfer of lateral forces and moments amongst the beams and columns framing into the joint. In a BCJ, the formation of plastic hinge and its location during a strong seismic event dictates the integrity of the moment resisting frame. In older constructions, prior to the enforcement of seismic detailing of BCJ with shear reinforcement in the joint, the formation of plastic hinge at the core of the joint, led to large deformation at the core and a brittle failure of the frame. In the recent years, these deficient BCJ are being retrofitted using carbon fibers reinforced plastic (CFRP) sheets. These sheets provide enhanced strength and stiffness to the joints, precluding the brittle joint failure. CFRP strengthening shifts the formation of plastic hinge from joint core to the beam, transforming it to strong column-weak beam joint. The modern code requirement enforces the strong column-weak beam joint philosophy by providing shear reinforcement in the joint to ensure a ductile moment resisting frame.

The location of plastic hinge in the beam both in older beam column joints with CFRP retrofit and new constructions with shear reinforcement is of critical importance. In many cases with CFRP retrofit and shear reinforcement, the plastic hinges are formed at the face of the column at the BCJ interface. In severe seismic events the plastic hinge at the joint may penetrate into the core of the joint, thereby degrading the joint strength and stiffness and loss of bond between the bars and the concrete. The effectiveness of the plastic hinge in dissipating the energy is compromised and brittle failure at the joint can ensue. It should therefore, be an important objective in retrofitting
BCJ with CFRP and shear reinforced joints in modern construction to ensure that the plastic hinge is developed at a reasonable distance away from the column face, well into the beam to ensure that the plastic hinge does not propagate into the joint under cycles of earthquake loading.

This paper presents the results an experimental program in which the CFRP strengthening was carried out for a beam-column joint, in which shear failure in joint is precluded, to ensure the formation of plastic hinge well into the beam away from the BCJ interface. Finite element simulation of a CFRP strengthened exterior beam column joint was carried out using commercial software ABAQUS, which captured the experimental response.

**PLASTIC HINGE FORMATION IN BEAM COLUMN JOINTS**

Many of the buildings built around the world in the 1980’s were mostly designed to support gravity load and these buildings suffered from extensive damage at joints resulting in their partial collapse as witnessed in the recent earthquakes in Nepal 2015, Chile 2015, Japan 2010, Taiwan 1999, and in Turkey 1999. In the past three decades, a number of researchers and research groups have committed significant efforts into studying the behavior of BCJ under shear reversals, as well as on the development of design recommendations to ensure adequate connection behavior in reinforced concrete frame structures that are designed for large inelastic deformations. Design codes have evolved over the years to improve the capacity of non-seismically designed building joints to ensure a ductile behavior, such as ACI 352 (ACI Committee 352 2002).

Ductile mode of failure is desirable in structures where plastic hinges are formed before the overall failure of the structure, but the location of the plastic hinges is of great importance to prevent its further penetration into regions with lesser capacity. In ductile moment resisting frames, which comprises of beam and columns, plastic hinges are usually formed at the interface of the BCJ, especially for short span beams, but in long span beams where the dominant load is gravity load, the plastic hinge may form outside the BCJ interface at a distance inside the beam (Paulay and Priestley, 1992). When the beam top or bottom reinforcement yields at the interface of the BCJ, the joint experiences severe shear cracks as a result of deterioration of the bond between the steel and the concrete, and thereafter leads to shear failure. To avoid the formation of plastic hinge at BCJ interface, several approaches have been proposed, which are based on the same principles. The first was proposed by Paulay and Priestley (1992). It involves a special detailing of longitudinal steel reinforcement in the beam at critical section around the joint. Fadi et al. (2014) proposed slotted-beam detailing technique for ensuring plastic hinge relocation from the face of the column to the beam. Reversed cyclic load testing experiments on BCJ with slotted-beam at the interface of the BCJ, and at 300mm away from the BCJ interface showed the viability of this approach in new constructions.

In shear reinforced BCJ, wherein the plastic hinge is formed at the BCJ interface, a proper web bonded or flange bonded CFRP can be implemented to ensure that plastic hinge formation is translated from the interface to an appropriate distance away from the interface. Mahini and Ronagh (2009, 2010, 2011) proposed web-bonded CFRP retrofitting technique for exterior beam column joints with shear reinforcement in the joint, for enhancing the strength and ductility of the joints and relocation of plastic hinge. The BCJ was designed in compliance to new seismic design code with shear reinforcement provided at the joints to preclude brittle joint failure and ensure that the plastic hinge is formed at the interface of the BCJ. Application of three layers of web-bonded CFRP sheets (0.495 mm thick) extended into the beam resulted in the translation of the plastic hinge from the face of the column to 150-300 mm away from the BCJ interface into the beam. Finite element simulation of the joint using the software ANSYS captured the response and the plastic hinge formation with a good accuracy. Dalalbashi et al. (2012) verified the experimental results of Mahini, and numerically investigated the efficiency of flange-bonded CFRP sheets in plastic hinge relocation in RC joints. The problems and difficulties encountered in retrofit of joints using web-bonded CFRP sheets were solved by using flange bonded FRP. Experimental study on the appropriate anchorage system for flange bonded CFRP in retrofitted reinforced concrete BCJ with the aim of plastic hinge relocation was carried out by Esfami and Ronagh (2013). There was a remarkable improvement in the load carrying capacity and elastic stiffness of the CFRP-retrofitted specimens, which confirmed the efficiency of the suggested system. Furthermore, the plastic hinge was relocated away from the beam-column interface.

In buildings constructed prior to the enforcement of joint shear reinforcement, the joints were mostly deficient, resulting in the formation of plastic hinge in the core of the joint as shown in Figure 1(a) (Baluch et al. 2012), which led to brittle failure. In cases where the plastic hinge is formed at the BCJ interface as shown in Figure 1(b) (Baluch et al. 2012), the penetration of yield surface into the core resulted in the failure of the joint. Several strategies for repair and strengthening of deficient beam-column joints in buildings have evolved over the years with Carbon Fiber Reinforced Polymer (CFRP) sheets and laminates being a widely adopted technique. Major repair and strengthening of earthquake damaged structure and enhancement of non-seismic designed and detailed BCJ in the recent years were carried out using CFRP. High tensile strength (both static and long term), low weight, easy
application and corrosion resistance of CFRP composite has made it a preferable choice for use in repair and strengthening of BCJ.

The strengthening technique using CFRP has successfully precluded the failure of the joint in shear, with the mode of failure changing from a brittle joint failure, which results in extensive damage to the structure to a ductile failure mode with the plastic hinge forming at the beam-column joint interface.

![Figure: 1 Formation of Plastic hinges](a) At the core  (b) At BCJ Interface (c) At BCJ interface in CFRP retrofit (d) Well into the beam in CFRP retrofit

Web bonded CFRP sheets in diagonal configuration were used by Halahla (2014) for retrofitting exterior BCJ deficient in shear. The brittle shear failure of joint and plastic hinge formation in the joint was precluded together with an enhancement of load carrying capacity by 13%. The failure mode of the CFRP retrofitted joint changed from brittle joint failure in shear to ductile beam failure with the plastic hinge forming at the BCJ interface, as shown in Figure 1(c), without significant cracks in the joint. The formation of plastic hinge at the beam column joint interface, although it prevents severe damage and collapse of the building, it may preclude the repair and rehabilitation of the building. It is therefore, important to adopt a CFRP strengthening strategy in which the plastic hinge during a seismic event is formed away from the BCJ interface, well into the beam. This will ensure a localized element level failure in beam which can be easily repaired and strengthened. Figure 1(d) shows a typical scheme of CFRP strengthening which results in the formation of the plastic hinge well into the beam at the point of truncation of the CFRP sheet (Mosallam et al., 2014).

EXPERIMENTAL INVESTIGATIONS

Details of Tests Specimens

The specimens tested in the experimental program are representative of a typical exterior beam column joint in a moment resisting frame which was fabricated using concrete with joints having no transverse reinforcement. The specimens were tested under monotonic loading to failure. The specimens designated as control sample (CS), and retrofitted sample (RS) have 250 mm x 300 mm columns and 250 mm wide x 300 mm deep beams designed with 3-16 mm diameter flexural reinforcement in the beams to generate flexural failure in the beam. The geometric configuration and reinforcement details of the test specimens are shown in Figure 2(a). Compressive strength tests and split cylinder tensile strength tests were conducted on 75 x 300 mm cylinders. The average 28-day compressive and tensile strength of the concrete used in the BCJ were 50 MPa and 2.5 MPa respectively. The 50 MPa concrete ensured that there was no shear failure in the joint due to penetration of yield zone into the joint. The average values of yield and ultimate strength of the longitudinal and stirrup reinforcements as measured experimentally was 610 MPa. The specimen RS was retrofitted with CFRP sheet attached to the flange of the BCJ as shown in Figure 2(b). The edges of the longitudinal applied CFRP sheets were wrapped with small width transverse CFRP sheets to prevent peeling of the longitudinal CFRP sheets at its edges. Sika Hex 230C unidirectional CFRP sheets was used for retrofitting the beam column joint, with a two part SIKADUR 330 epoxy, using the wet-lay procedure. The properties of CFRP sheets used for retrofitting the BCJ, as provided by the manufacturer, are shown in Table 1. The properties of SIKADUR 330 glue (a 2-part epoxy impregnation resin) are shown in Table 2.

Experimental Setup, Instrumentation and Testing protocols

The BCJ were tested in a self-reaction steel loading frame consisting of axial and lateral loading systems. The loading frame with a specimen and testing arrangement is shown in Figure 3. Two hydraulic jacks were used for the application of axial and lateral loads. One 30-tons capacity hydraulic jack was used for the application of axial load on the column and the second hydraulic jack with 10-tons capacity was attached to the tip of the beam for applying lateral loading on the beam. Load cells, LVDT’s, and strain gauges were installed on the test specimens to monitor the loads, deflections and strains. A 20-ton load cell was installed at the top of the column to measure the axial load on the column and two 10-ton load cells were installed at the top and bottom of the tip of the beam.
to measure the lateral loads under monotonic push load (Figure 3(a)). Two LVDT’s were installed in the joint region to observe the diagonal crack openings and another two LVDT’s were also installed at the top and bottom of the column to monitor the movements. One string type LVDT (Patriot) was installed at the tip of beam to measure the deflections. Strain gauges were installed on the surface of reinforcing steel, concrete and CFRP to monitor strains. The columns were initially loaded monotonically with an axial load of 150 kN, which was kept constant in both retrofitted and normal BCJ, and then the BCJ were subjected to downward displacements applied at tip of the beam at each time step until the failure of the specimen.

Table 1 – Properties of CFRP Sheets.

<table>
<thead>
<tr>
<th>Mechanical Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>1.76 g/cm³</td>
</tr>
<tr>
<td>Thickness</td>
<td>0.13 mm</td>
</tr>
<tr>
<td>Tensile stress limit in Fibre direction (X_t)</td>
<td>715MPa</td>
</tr>
<tr>
<td>Compressive stress limit in Fibre direction (X_c)</td>
<td>15MPa</td>
</tr>
<tr>
<td>Tensile stress limit in transverse direction (Y_t)</td>
<td>23MPa</td>
</tr>
<tr>
<td>Compressive stress limit transverse direction (Y_c)</td>
<td>15MPa</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.2</td>
</tr>
<tr>
<td>Elastic Modulus in Principal direction 1 (E_1)</td>
<td>61GPa</td>
</tr>
<tr>
<td>Elastic Modulus in Principal direction 2 (E_2)</td>
<td>5.5GPa</td>
</tr>
<tr>
<td>Shear Modulus in principal direction 1 (G_{12})</td>
<td>2800MPa</td>
</tr>
<tr>
<td>Shear Modulus in principal direction 2 (G_{13})</td>
<td>2800MPa</td>
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<tr>
<td>Shear Modulus in principal direction 3 (G_{23})</td>
<td>1590MPa</td>
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<tr>
<td>Cross product term coefficient term f^*</td>
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<tr>
<td>Shear strength S_{12}</td>
<td>56MPa</td>
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Table 2: Mechanical Properties of Sikadur Epoxy

<table>
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<tr>
<th>Property</th>
<th>Value</th>
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<tbody>
<tr>
<td>Modulus of Elasticity</td>
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<td>Tensile Strength</td>
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<tr>
<td>Ultimate Strain</td>
<td>0.9%</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Figure 2: Specimen configuration (a) Control Specimen CS, (b) Retrofitting Scheme for Specimen RS.

**Flange bonded CFRP Retrofit of Exterior Beam Column Joint**

The BCJ specimen RS was retrofitted using SIKAK 230C CFRP sheets bonded to the flange of the BCJ on the top of the beam using SIKADUR 330 glue consisting of two parts, mixed in ratio 4: 1, using a wet lay procedure. Two layers of the CFRP sheet was wrapped on the beam-column joint, and 20 mm wide wraps were attached to the termination point of the CFRP sheets to ensure that the CFRP was properly held in place and do not peel at the edges. The CFRP retrofitted specimen RS is shown in Figure 3(b). The CFRP sheet applied to the BCJ with epoxy resin was allowed to cure as per manufacturer’s recommendations and then tested, under the monotonic displacement regime 7 days after the applications of CFRP.
RESULTS AND DISCUSSIONS

Failure mode and Ultimate Load of the control specimen (CS)

In the control specimen (CS), the flexural reinforcement in the beam experienced significant yielding resulting in flexural cracking in the beam, primarily at the BCJ interface. The specimen was tested under monotonically increasing load up to failure. The cracks in the specimen CS at the top of the beam are shown in Figure 4(a). The beam-tip load deflection curve for the specimen CS is shown in Figure 4(b). The first crack in the specimen was observed at a load of 36.1 kN at the beam-column joint interface. The crack at the BCJ interface became wider as the test progressed. Under increasing load, additional flexural cracks developed near the BCJ interface as shown in the figure. The flexural steel at the top of the beam yielded at a load of 105 kN after which the stiffness degraded and the deflection increased with small increase in the load. Very fine diagonal shear cracks were observed in the joint of the specimen during the experimental test. Horizontal cracks at the top of the joint also developed, which is probably caused by the penetration of yield into the joint. The yielding of the beam longitudinal reinforcement at the BCJ interface resulted in the formation of plastic hinge at the interface. The plastic hinge penetrated into the core of the BCJ and deteriorated the bond between the longitudinal steel and core concrete. The reinforcement yield penetration into the joint led to the termination of the test at an ultimate load of 115 kN at a displacement of 30.6mm.

Figure 4: (a) Failure of specimen CS (b) Load-displacement curve for CS
Failure mode and Ultimate Load of the CFRP Retrofitted Specimen (RS)

The flange-bonded CFRP retrofitted specimen RS behaved in a different manner compared to the specimen CS. The first crack occurred at the cutoff point of the CFRP on the beam at a load of 42 kN. Although, a minor flexural crack also occurred at the interface of the BCJ, and other parts of the beam, the crack at the CFRP cutoff point widened as the load increased. The bonding of the CFRP sheets on the flange of the beam at the critical section signified an additional reinforcement at that location which increased the section capacity, thereby making the section stronger. This resulted in the formation of the plastic hinge at the location where the CFRP sheets are truncated on the beam as shown in Figure 5(a). This phenomenon agrees to the reinforcement detailing of seismically designed frames suggested by Paulay and Priestley (1992). The required development length for the CFRP beyond the BCJ interface was achieved by bonding the CFRP laminate on the column, to avoid creating a groove at the BCJ interface as suggested by Eslami et al. (2013), which can be very difficult to achieve in real life strengthening of structures. Also, the grooving technique if not properly covered with good concrete can cause corrosion of reinforcement at the ingress of deleterious substances, especially in areas with harsh corrosion friendly condition. The effect of CFRP length was not considered in this study to verify the work of Eslami et al. (2013) and Dalalbashi et al. (2012). The CFRP length chosen in this study was taken as 1/3rd of the beam length, which in this case was 300 mm.

The longitudinal reinforcement in beam, in the specimen RS, yielded at a load corresponding to 114 kN with fine shear crack occurring at the core of the BCJ, which did not grow under increasing load. The yielding of bars in this specimen occurred at a higher load as compared to the control specimen CS, due to the presence of CFRP in the specimen RS. After the development of cracks at BCJ interface, CFRP sheet participates in resisting the applied loading. Yielding of the steel occurs at the location on the beam where the CFRP sheet is truncated (Figure 5(a). The specimen failed at a load of 145 kN and a beam tip displacement of 42.6mm. Comparison of the two specimens (CS and RS) showed a 26% increase in load carrying capacity, and better ductility, due to the formation of the plastic hinge away from the BCJ interface, well into the beam.

FINITE ELEMENT SIMULATION OF THE BEAM COLUMN JOINT

The commercially available nonlinear finite element software ABAQUS (2013) was used to simulate and validate the experimental results conducted on exterior BCJ. The concrete damage plasticity (CDP) model in ABAQUS proposed by Lubliner et al. (1989) was chosen. The CDP model has the capability to simulate the elastic and inelastic behavior of concrete in tension and compression with different yield stress as observed in the laboratory. The CDP model uses a yield surface for its failure criterion by combining the Drucker-Prager criteria with the Rankine criteria. To account for the complete behavior of concrete, ABAQUS requires the tension damage \( d_t \) and compression damage \( d_c \) parameters. The compression damage parameter \( d_c \) is related to the plastic strain \( \varepsilon_{pl} \) which is determined from the inelastic strain, and the tension damage parameter \( d_t \) solely depends on \( \varepsilon_{pl} \).

The steel reinforcement (longitudinal and transverse) is modeled as a truss element embedded in concrete assuming a perfect bonding between the steel and concrete, using the slave and master constraint so as to solve the problem.
with node compatibility during the meshing processes. Chen (1982) stated that if truss element is used for steel reinforcement, it is unnecessary to consider the complex behavior of steel when subjected to multi-axial stresses. Therefore, a perfect elasto-plastic material constitutive law is used to idealize the stress-curve under tension and/or compression.

The behavior of FRPs depend on their mechanical properties, fiber orientation, length, shape and constituent fibers, properties of the epoxy adhesive, and nature of the surface to which they are bonded. FRPs usually display linear elastic property until failure without any inelastic deformation. There are many proposed theories to represent the orthotropic plane stress failure of FRPs. In this study, the Tsai-Wu theory was used to simulate the failure surface of CFRP in the ABAQUS program because it is suitable for unidirectional FRP as compared to the Tsai-Hill’s criterion as it accounts for Bauschinger’s effect (Ramesh, 2014). Tensile values were taken as positive, while compressive taken as negative so as to account for FRPs behavior in both tension and compression as required in Tsai-Wu theory. Hadigheh et al. (2012) used ABAQUS FEM simulation to verify the performance of weak-beam, strong column RC frames strengthened at the joints by FRP. The epoxy was modeled as contact element and Hillerburg’s fracture energy criterion was used. The numerical result obtained was in excellent agreement with the experimental results.

Many researchers including Carlos et al. (2005), Wein et al. (2015), Priya et al. (2014) and Gopinath et al. (2014), have studied the effect of using contact element at the interface of the FRP and concrete to simulate the real life scenario in numerical modeling. Cohesive zone model (CZM) has been used by some of the authors by using the traction-separation law. It has been reported that the contact element is not a key factor for predicting behavior of the FRP-concrete bond, but the fracture energy of the concrete-epoxy interface plays an important role in the results of such prediction. Hence, many authors have assumed a perfect bonding between the FRP and concrete interface. Despite the foregoing, in this study, the epoxy bond was modeled using a plain strain element (CPE4R), since the epoxy has two surfaces: concrete-epoxy surface, and epoxy-FRP surface. The epoxy was bonded to the concrete surface using ties to ensure that nodes on the interface of the epoxy have the same displacement as nodes on the concrete surface. By doing this, normal and shear stresses along the concrete epoxy-interface would have been accounted for. The properties of epoxy SIKADUR 330 used in the FE modeling are shown in Table 2. The finite element model of the experimental beam column joint specimen in ABAQUS is shown in Figure 6(a). The control specimen was used for verification of the numerical scheme using appropriate mesh size, and termination time. Typical damage patterns in the control specimen CS and the CFRP retrofitted specimen RS are shown in Figure 6(b) and Figure 6(c).

![Figure 6. FE Modelling in ABAQUS: (a) FE Model of the BCJ (b) Damage in specimen CS, (c) Damage in specimen RS.](image)

The results obtained from the simulation of the BCJ in ABAQUS showed a similar trend as noticed in the experimental study. The ultimate failure loads and beam tip displacement for the control specimen CS from FE simulation are 116 kN and of 35 mm respectively. For the flange-bonded CFRP sheets specimen RS, the ultimate failure loads and beam tip displacement are 153 kN and 47.8 mm respectively. The load-deflection curve for both experimental and FE simulation are shown in Figure 7a and Figure 7b. Numerical results capture the experimental data in both specimens with a good accuracy. The strains in the concrete and beam longitudinal reinforcement for both experimental and numerical studies were also found in agreement.
CONCLUSIONS

This paper reports the findings of experimental and numerical investigations for relocating plastic hinges away from the interface of a non-seismically designed exterior BCJ. Two layers of externally bonded unidirectional CFRP sheet, 300 mm in length, with CFRP width covering the beam width was used for retrofitting the BCJ specimen which was tested under displacement control monotonic loading. Finite element simulation of the BCJ was carried out using the commercial software ABAQUS. It was observed that the retrofitted samples had higher ultimate capacity, higher ductility, and plastic hinge was relocated from the BCJ interface to the cutoff point of the CFRP sheets on the beam. This relocation of plastic hinge is desirable for better performance of RC frame structures subjected to lateral seismic loads, which places higher demand on the joint. The numerical results were in very good agreement with the experimental work as they give almost the same load capacity, stiffness and beam tip displacement. The finite element simulation in ABAQUS incorporated the CFRP-Epoxy-Concrete bond, material constitutive model for the FRP and fracture energy of the fiber/epoxy, which gave very close results. Further studies are being conducted by the authors to assess the response of the flange bonded CFRP retrofit and its effect on plastic hinge relocation under cyclic loading.

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RESILIENCE RESERVE OF RC COLUMNS EXTERNALLY CONFINED WITH COMPOSITE ROPE OR FRP SHEET UNDER SEISMIC OVERLOADS

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ABSTRACT

This paper highlights unique inherent seismic resilience aspects of reinforced (RC) columns externally confined with composite rope or fiber reinforced polymer (FRP). It discusses the inherent seismic resilience reserve of this subsystem at material, at section and at structural member level, both qualitatively and quantitatively from a structural point of view. The investigation focuses on the damage control at member level that may prevent collapse of critical infrastructure. A design concept is concluded towards enhanced inherent resilience of such subsystems from a structural point of view. This concept concerns subsystems in which the weakest link (in our case also being the main bearing material, the concrete core) suffers a local damage initiation (susceptible to further accumulation) that disrupts its uniformity and homogeneity in response. In that case, adequate confining action may maintain damage-sensitive-restriction in a way that globalizes damage (damage redistribution) inside the core and makes the core again more uniform and more homogeneous in its response.

KEYWORDS

FRP, RC column, composite rope, strengthening, confinement, damage, resilience, analytical model.

INTRODUCTION

Resilience is a multidisciplinary and interdisciplinary concept defined as “the ability to prepare and plan for, absorb, respond, recover from, and more successfully adapt to adverse events” (NAC 2012). It covers different fields such as ecology (Holling 1973), materials science and disaster mitigation. As far as the built environment is concerned, there is a need to improved performance during and after a disruptive hazard event (McAllister 2013) and metrics need to be developed to account for resilience of built environment at the level of components, infrastructure systems and whole communities. The current study is limited to earthquake induced hazard and highlights the resilience reserve of reinforced concrete (RC) columns retrofitted with composites.

Different alternatives in design of new structures or of retrofit techniques in seismic redesign of existing structures and of their structural components, may affect significantly the resilience at infrastructure and at community level. Recently, Biondini et al. (2015), presented a probabilistic analytical approach to assess the seismic resilience of concrete structures exposed to chloride-induced corrosion. They concluded that time-dependent effects of corrosion may greatly reduce functionality and resilience of the structure during a seismic event. Functionality of the structure is associated with its seismic capacity. Their research indicates the importance of including in the seismic design of resilient structures all potential hazards for the whole life-cycle. In another study, Echevarria et al. (2015) highlighted the improved resilience of critical bridge infrastructure when concrete-filled fiber reinforced polymer (FRP) tube (CFFT) columns are used, instead of RC ones. They performed a comprehensive experimental study of columns suffering blast, fire or earthquake hazards (multi-hazard experimental assessment) of different intensity and concluded lower damage of the columns, lower restoration time and lower repair costs for the case of CFFT. Of course, multilevel societal needs and potential, need to be satisfied through the development of risk-based performance goals (McAllister 2013). In that aspect, Bocchini et al. (2014) proposed a risk assessment approach to unify resilience and sustainability requirements.

The current study highlights some aspects of comparative qualitative and quantitative assessment of the inherent structural resilience of RC columns of existing structures retrofitted with composites under seismic excitations. The investigation is limited to the performance of the columns to maintain their axial capacity. Echevarria et al. (2015), acknowledge the criticality of this issue as in example, after an earthquake a bridge may be still in service to provide first responders and emergency vehicles access to exclusive regions affected. This may have a great
impact on resilience at structural and societal level. Similarly, RC critical infrastructure such as Hospitals, Fire Stations and Command Centers, etc may require similar performance levels as a whole in order to operate. In the current study the retrofitted RC column is considered mainly as a resilient subsystem, being critical component of structure that belongs to a network of critical infrastructure to fulfill resilience requirements. However, the investigation discusses also resilient aspects of the materials and sections within the subsystem.

**STRUCTURAL INHERENT RESILIENCE**

At retrofitted RC column level, the resilience may be interpreted as the ability to: absorb, resist, recover from and more successfully adapt to seismic overloads or overdisplacements with respect to ultimate limit states required by design. This study is limited to cases of columns redesigned to overcome above challenges with increased displacement ductility, through externally confined concrete. In general, increased strength, or passive control or seismic isolation could be realized instead in redesign of structures. Residual axial load capacity at high displacement ductility levels requires the suppression of fragile-type related failures such as: concrete shear failure, concrete cover spalling, concrete compression, steel bar premature buckling under compression, relative slip of steel bars at lap splices and FRP fracture in order to ensure steel yielding and full utilization at hardening stage up to emergence of global instability issues.

Figure 1. RC (or plain) concrete columns with PPFR passive or active confinement

**Composite Rope Confinement Effects on Structural Resilience**

*Fiber rope confined columns*

Composite rope wrapping may succeed in the above tasks through adequate external confinement. Furthermore, adequate composite rope wrapping ensures maximum utilization of concrete axial strain potential, as it allows for lateral strain redistribution according to varying demands of the column (affected by the concrete cover, confined core, steel stirrup yielding, bar buckling and section shape) throughout loading. Redistribution of strains causes redistributed damage and thus extensive and uniform cracking of concrete. This unique characteristic at section level contributes to enhanced resilience of the column being under axial loads, during seismic excitations (Rousakis 2014, Rousakis 2013, Rousakis and Tourtouras 2014). Figure 1 shows RC column 500PPL4 from Rousakis and Tourtouras (2014), wrapped with 4 layers of polypropylene fiber rope (PPFR). The column exhibits extensive global buckling at member level, multiple local bar buckling, multiple local bulging of the concrete core and intact composite rope, while maintaining the axial load bearing capacity (see Figure 1). This test was early stopped as depicted in Figure 1, carrying ever-increasing axial load at 0.06 concrete axial strain. Similarly, the tests of columns without steel reinforcement (PPL4) or of RC column 220PPL3P1.075% with pretensioned PPFR at 1.075% strain were early stopped without failure of the retrofitted region. The pretensioned column exhibited prolonged elastic behavior and higher bearing stress at similar strain levels than 500PPL4 despite having only 3 layers of wrapping. Higher pretension of 4 layers of PPFR at 2.65% strain (column 500PPL4P2.65%) provided
Further upgrade but the test was early stopped because of undesirable failure at the weaker, non-pretensioned base region of the column (outside the control region).

Even in cases with lower deformability basalt composite ropes (Rousakis et al. 2015), the achieved concrete axial strain ductility provides remarkable resilience as it surpasses the practical requirements in any case (higher than 0.07 strain). The unique characteristics of adequate composite rope wrapping, suppress common but low resilience failures observed in cases of concrete columns with internal steel stirrup or with external FRP jacket or with internal steel and external FRP confinement under severe axial loading that may lead to main shear-related cracks that further evolve and exacerbate non-uniformities. These irrecoverable non-uniformities initiate upon yielding of the steel stirrups or upon fracture of fibers of FRP jacket and the damage accumulates locally, leading to global column failure. In terms of axial load – axial deformation, the enhanced resilience of the column wrapped with composite rope is revealed with the temporary axial load drop of the column. Even in cases of very low rope confinement, the severe load drop may reveal a significant resilience potential of the composite rope to resume the axial load capacity to very high levels (see in example column 20PPL3R1 in Rousakis, 2013). Furthermore, in cases of fractured vinylon fiber ropes, the strand by strand fracture and FR-concrete friction provides remarkable levels of load recovery during successive cycles (see in example column VinL1v2R2 in Rousakis 2013). In cases of RC columns, the temporary load drops are even lower than corresponding plain concrete columns wrapped with composite rope. Resilience is owed to concrete deformability, rope strain redistribution, rope high deformability non-fractured, very low sensitivity to cracked-concrete-induced FR damage, strand-by-strand fracture of lower deformability rope. Maximum utilization comes with ‘spring-like’ response of bearing concrete at very high strain levels that denote significant degradation of the core “cohesion” through extensive cracking towards maximum homogeneity in response (Rousakis 2014).

The study by Kwon et al. (2016) verifies the efficiency of composite rope (or tape) reinforcement with very low effective normalized axial rigidity to enhance the lateral displacement performance of 2-column RC frames with inadequate steel stirrups (sparse stirrup spacing, low shear capacity, premature bar buckling). They use only one layer of Velcro tape of 0.3 mm thickness, made of nylon with self-anchoring detailing. This negligible Velcro confinement leads to negligible stress and strain enhancement of confined concrete section which has dimensions 200X300 mm and zero corner curvature radius. The provided confinement is at least 6 times lower than the one provided for column 500PPL4 in Rousakis and Tourouras (2014). Yet, the elastic non-bonded composite rope reveals its unique confining characteristics by restricting concrete cover spalling and delaying damage evolution, leading to increased ultimate displacement of the frame by factor of 1.25 and to increased absorbed energy by factor of 3.7 (columns VARC).

**Effects of composite rope confinement on P-δ response of RC columns**

In what follows the experimental behavior of the retrofitted RC column provided by Kwon et al. (2016) serves as the basis for comparative elaborations. The analytical σ-ε curve of the VARC column provided in Kwon et al. (2016) study under concentric axial load, is generated with Rousakis and Tourouras 2015 model (see Figure 2). This iterative model is based on Spoelstra and Monti (1999) model (fib 2001), further developed to take into account dual steel stirrup – FRP jacket effects, steel bars and pretensioned confinement. Of course, recently published studies may allow for further development of the model to incorporate additional features (see Wu and Wei 2014, Nistico et al. 2014, Wu and Wei 2015 and Li and Wu 2015 among others). In Figure 2, it is shown that the difference in terms of σ-ε between the non-retrofitted (ORC) and the retrofitted column (VARC) is negligible. The maximum effective retrofit confinement rigidity ratio (that is the maximum effective lateral stress (σjmax) divided by the lateral strain (εl) and multiplied by the effectively confined concrete core coefficient (αj) and divided by the plain concrete strength (fco) namely (fjmax/εl)*αj/fco is 0.25 and may be utilized as a measure of provided external confinement among different columns.

The corresponding confinement rigidity ratio for column 500PPL4 is 1.53 and the σ-ε curve is also shown in Figure 2. The σ-ε of the VARC column under compression with confinement rigidity 1.53 (equal to 500PL4) is shown as VARC1.53. The curve has descending post-maximum branch as the existing steel stirrups have lower efficiency than in 500PPL4 column. Moreover, degrading stress-strain behavior is commonly met in real-size RC columns. However, the load degradation for VARC1.53 is better controlled. Another interesting column under axial
compression is the specimen R-1-8H with dimensions 508 mm by 737 mm with post-rounded corners to radius of 25.4 mm (De Luca et al. 2011). It is wrapped with 8 layers of hybrid glass – basalt FRP sheet and the corresponding confinement rigidity ratio for column R-1-8H is 2.53. Its $\sigma$-$\varepsilon$ curve according to Rousakis and Tourtouras (2015) model is presented in Figure 2 as VARC2.53. There, the descending branch is further controlled and stabilized at a higher bearing stress, while the axial strain ductility of confined concrete is expected higher than 0.07 (see experiments in Rousakis and Tourtouras 2014). Yet, based on the experimental performance of the FRP confined column R-1-8H, the axial strain ductility of the VARC column would be very low if FRP retrofitted, because of the low deformability and axial rigidity of the FRP jacket and because of the zero corner radius.

![Figure 2. RC (or plain) concrete columns with PPFR passive or active confinement](image)

![Figure 3. Pushover analyses of VARC columns for different confinement rigidity ratios](image)

The predicted displacement at failure based on the pushover analysis of the columns VARC with constant axial load, using RESPONSE 2000 software (see also Rousakis et al. 2016) is 87.2 mm (experimental values are 86.9 mm or 95.4 mm). In the case the column is retrofitted with 6 layers of Velcro that equals the confinement rigidity ratio of column 500PPL4 (1.53) the analytical displacement at failure of the RC columns VARC1.53 is 107.6 mm.
(23% higher displacement). In the case the column is retrofitted with 10 layers of Velcro that equals the confinement rigidity ratio of column R-1-8H (2.53), the analytical displacement at failure of the RC columns VARC2.53 is 110 mm (26% higher displacement). Figure 3 shows all analytical curves. It is obvious that given the zero corner radius of the cross section of the VARC column, the efficiency of the FRP retrofit will be negligible as the reinforcing fibers will suffer premature failure. Even if the corner radius was adequate, the very low confinement provided, would yield similar results.

CONCLUSIONS

The unique features of RC columns confined with nonbonded composite ropes may contribute to a conceptual performance-based design towards enhanced inherent resilience of systems that contain a weak component susceptible to fatal damage accumulation. Enhanced inherent resilience of such systems may be achieved with uniform redistribution of damages inside the main bearing core mass, by peripherally confining the core through contact (not fixing) with elastic material (that does not accumulate damage). This material needs to be highly deformable and flexible (low sensitivity to damage acting as barrier) so that can locally deform and thus redistribute strains around and inside the core. In that case, total core mass under confining action is always the maximum one and the confining action is ever-increasing for higher axial compression of the core, imposing the core to higher multiaxial compression. This concept ensures that even if the weakest link (and main bearing material, the concrete core) suffers a local damage initiation (susceptible to further accumulation) that disrupts its uniformity and homogeneity in response, the confining action maintains damage-sensitive-restriction in a way that globalizes damage inside the core and makes the core again more uniform and more homogeneous in its response. In that case the energy dissipated by this subsystem is maximized.

In composite rope wrapped RC columns under cyclic axial load (i.e. induced with seismic excitations), the local concrete damage, the extensive steel bar local instability and concrete global instability is self-redistributed (inherently, if considered at structural member system level) and ever-increasing bearing axial load results (multiple buckled bars and multiple buckling modes at member level). Maximum utilization comes with ‘spring-like’ response of bearing concrete that denotes significant degradation of the core “cohesion” through extensive cracking towards maximum homogeneity in response. The concrete axial strain ductility of the column is remarkably high as well as the dissipated energy.

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ANALYSES OF PLASTIC HINGE REGIONS IN FRP-CONFINED RC COLUMNS UNDER CYCLIC LOADING

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ABSTRACT

Fiber-reinforced polymer (FRP) jacketing of reinforced concrete (RC) columns is always considered as a very effective method to enhance the seismic performance of existing RC columns. The investigation on the plastic hinge behaviour of FRP-confined RC members has largely been experimentally based because of the high complication of the problem. Due to large number of variables involved and the high cost of testing, different works reported in the literature could only focus on a certain aspect or special cases and different even contradictory outcomes have often been reported. The present paper intends to investigate the behaviour of plastic hinge of FRP-confined RC columns numerically using three-dimensional finite element method (FEM). Lengths in the plastic hinge zones, involving rebar yielding zone, concrete crushing zone, and curvature localization zone, of FRP-confined RC columns subjected to monotonic and cyclic loading are firstly investigated in detail. It is found that all three zones are significantly affected by the loading scheme. Effect of FRP jacketing length on the length of concrete crushing zone are also analyzed. It is found that, for flexural retrofitting of RC columns, FRP jacketing length needs only to cover the concrete crushing zone, which is closely related to the FRP confinement ratio.

KEYWORDS

Plastic hinge, FRP, RC columns, confinement, FEM.

INTRODUCTION

In recent years, fiber-reinforced polymer (FRP) jackets have become popular in providing confinement to earthquake resistant reinforced concrete (RC) columns (Ibell et al. 2009). A sufficient ductility for RC columns can be achieved by FRP confinement at a potential plastic hinge region (Seible et al. 1997; Saadatmanesh et al. 1997). The plastic hinge region is defined as the physical region over which the member experiences inelastic deformations and severe damage. The plastic hinge length is critical not only for the design of new members but also for the seismic retrofitting of old structures.

Due to high nonlinearly of materials and complicate interactions between constitutive materials involved, most researchers studied the plastic hinge of RC members by experimental study and a few researchers studied the problem through numerical modeling. Zhao et al. (2012) studied the plastic hinge regions in RC beams under monotonic loading by finite element method (FEM) and a systematic parametric studies on the influence of plastic hinge length of full-scale square RC beams was carried out. Youssif et al. (2015) recently investigated the plastic hinge behavior of FRP-confined circular RC columns under monotonic loading by numerical simulations and an analytical model for ultimate drift ratio was proposed. The previous numerical studies on plastic hinge have been performed under monotonic loading and there are few studies investigating cyclic behavior of plastic hinge in FRP-confined RC members. Unlike monotonic behaviors, the cyclic behaviors of rebar yielding development, the stiffness degradation of confined concrete, as well as the bond-slip between steel and concrete are believed to have some influences on the development of plastic hinge zones of FRP-confined RC columns. This gives rise to the need for more comprehensive studies of this problem.

This paper aims to investigate the plastic hinge problem of FRP-confined RC columns in great detail through finite element simulations. The FEM model is first calibrated with test results. After that, a systematic study was
subsequently carried out to investigate the plastic hinge regions of FRP-confined RC columns subjected to monotonic and cyclic loading. Finally, the minimum FRP jacketing length for flexural retrofitting of RC columns with various confinement ratios is analysed.

**FINITE ELEMENT MODELLING AND IMPLEMENTATION**

General finite element software ABAQUS (2010) is used in this work. The constitutive models involve concrete, steel reinforcement and interfaces between concrete-steel reinforcement. Since the confinement effect of FRP jacket is embodied in the stress-strain relationship of concrete, FRP jacket is not created in FEM model. Detailed modeling and the parameters are summarized below.

The 8-node linear brick element C3D8R in ABAQUS is employed for concrete elements. The mesh size of the concrete element for RC columns is 20 mm. Elements smaller than 20 mm make little difference to the results, according to the mesh convergence study.

Damaged plasticity model is adopted to model the material behavior of concrete. The ascending section of the tensile relationship is assumed to be linear and softening is assumed to be exponential. The fracture energy method is adopted for post-peak cracking of concrete. The strain at peak stress $\varepsilon_{p,c}$ is assumed to be 0.0001. The fracture energy for concrete $G^f_c$ is assumed to be 170 N/m (Hillerborg 1985). The unloading and reloading paths of concrete in tension are taken to be linear and pointing towards the coordinate origin.

In compression, the analysis-oriented stress-strain model by Teng et al. (2007) is used to model envelop curve of both unconfined and confined concrete. The unloading and reloading paths of concrete are taken to be linear and pointing towards the zero stress points with residual strains. The modulus of the unloading and reloading paths of unconfined concrete is determined by the damage model proposed by Lubliner et al. (1989), where the damage factor is defined by:

$$d_c = 1 - \frac{\sigma_{un}^{\text{un}}}{f_c}$$

where $f_c$ is the compressive strength of concrete, $\sigma_{un}^{\text{un}}$ is the compressive stress corresponding to unloading strain.

For FRP-confined concrete, the compressive damage factor can be defined by

$$d_c' = 1 - \frac{\sigma_{un}^{\text{un}}}{E_0\left(\varepsilon_{un}^{\text{un}} - \varepsilon_c^p\right)}$$

where $\varepsilon_{un}^{\text{un}}$ and $\sigma_{un}^{\text{un}}$ are the unloading compressive strain and the corresponding stress; $E_0$ is the elastic modulus of concrete; $\varepsilon_c^p$ is the residual plastic strain during unloading and its relation to the unloading strain $\varepsilon_{un}^{\text{un}}$ can be expressed by the model proposed by Li and Wu (2015).

A 2-node truss element T3D2 in ABAQUS is employed for steel reinforcement. The mesh size is also assumed to be 20 mm. The bilinear model which considers strain hardening is adopted to model the stress-strain relationship of the steel reinforcement under monotonic load and a model proposed by Clough (1966) is employed to represent the unloading and reloading paths.

The Spring2 element is selected to define the nonlinear behavior between concrete and longitudinal tensile reinforcement. The concrete nodes and steel nodes with the same coordinates are connected by springs. A large linear stiffness is applied in the normal direction of the interface, while the unified bond-slip relationship proposed by Wu and Zhao (2012) is applied in the tangential direction to consider the nonlinear slip between concrete and steel bars. The bond stress-slip relationship can be transformed into the nonlinear spring force-relative displacement relationship in ABAQUS.

**MODEL VERIFICATION**

Two square RC columns (one with FRP confinement and another without), are simulated herein using the above described FEM model. Specimens Unit 3 (Ang 1981) and ASC-6NS (Iacobucci et al. 2003) were tested under constant axial loading and cyclic lateral deformation. It can be seen from Figure 1 that the unloading/reloading stiffness and load carrying capacity from the numerical results match well with those of measured results. Reasonable agreements are observed between measured hysteresis curves and FEM results.
The physical plastic hinge involves three different regions: the rebar yielding zone, concrete crushing zone and curvature localization zone (Zhao et al. 2012). In this section, all three regions are investigated in detail to get insight into the real plastic hinge zone of FRP-confined RC columns. Two specimens, one subjected to monotonic lateral loading and another subjected to cyclic lateral loading with fixed displacement increment of 1% drift ratio, are selected for analysis. The lateral displacement is increased under a constant axial force. The axial force is fixed to be 0.2N, where \( N = f_s A_s \) in which \( f_s \) is the compressive strength of concrete and \( A_s \) is the gross cross-sectional area of the column. Both specimens are confined with one layer (\( t_f = 0.167 \) mm) CFRP jacket with elastic modulus \( E_f \) of 245 GPa and fracture strain \( (\varepsilon_f) \) of 0.0171 and they have the same geometric dimensions and material properties. The specimen sizes are 400 x 400 x 1700 mm connected to a 600 x 800 x 1400 mm stub (Figure 2). The corner radius of the columns is 40 mm and the effective cantilever length is 1600 mm from the center of the loading to column base. Due to the symmetry of the column, half of the column is modeled. The compressive strength of unconfined concrete is 30 MPa. For both columns, steel bars of 20 mm diameter are employed as the longitudinal bar and steel bars of 10 mm diameter and spacing of 100 mm are adopted as transverse reinforcement. The steel has a yield strength of \( f_y = 460 \) MPa, an ultimate strength of \( f_u = 600 \) MPa and elastic modulus of \( E_s = 200 \) GPa. The reinforcement details and geometric dimensions of the columns are as shown in Figure 2.

**Investigation of rebar yielding zone**

The regions where the strain of tensile reinforcement has reached or exceeded its yielding value is defined as the rebar yielding zone. Figure 3 shows that although the length of rebar yielding zone varies as the deformation increases, the maximum length of this region \( L_{sy} \) from the column base is limited to a certain value. This observation is consistent with the test results in literature (Scott 1996).

It is also found from Figure 3 that \( L_{sy} \) of column under cyclic loading is larger than that under monotonic loading. The plastic hinge formation process under these two loading conditions are evidently different. Under monotonic loading, the length of rebar yielding zone increases significantly with the increase of the column deformation after it yields and keeps almost constant after the drift ratio of 3%. By contrast, under cyclic loading, evident increase in the length of rebar yielding zone is observed at each new displacement cycle and it stabilizes until the drift ratio of 5%. Residual stresses exist in longitudinal reinforcement at the beginning of new displacement cycles, which makes larger area of longitudinal reinforcement enter into the yielding phase, resulting in a larger \( L_{sy} \).

**Investigation of concrete crushing zone**

Figure 4 Concrete compressive strain distribution: (a) under monotonic loading and (b) under cyclic loading

To study the damage zone of FRP-confined RC column, the lengths of compression region \( L_{cs} \) and \( L_{cc} \), are investigated, where \( L_{cs} \) is defined as the length of the region where the concrete compressive strain is larger than the strain at peak stress of unconfined concrete (0.002) and \( L_{cc} \) is defined as the length of the region where the compressive strain is greater than the crushing strain of unconfined concrete (0.006). Careful examination of numerical results in Figure 4 shows that \( L_{cs} \) stabilizes at a certain value after the drift ratio of 2% while \( L_{cc} \) keeps increasing as displacement continues to increase. It can be also found from Figure 4 that both \( L_{cs} \) and \( L_{cc} \) for column under cyclic loading are significantly larger than those of column under monotonic loading. Compared with
monotonic loading, the stiffness degradation of concrete under cyclic loading makes it easier exceed the specified strain values (0.002 or 0.006), resulting in a larger \( L_{cs} \) and \( L_{cc} \).

Figure 2 Specimen details (unit: mm)

Investigation of curvature localization zone

Figure 5 shows the curvature distributions at different deflections beyond yielding for FRP-confined RC columns. It can be seen that the curvature varies significantly along the column length due to concrete cracking. After a column yields, most of the plastic curvature gets concentrated within a certain zone. A dividing point exists from which to the column base the curvatures increase rapidly while those outside the region remain almost constant. This numerical result is consistent with experimental observations reported in literature (Hines et al. 2004; Gu et al. 2010). The distance from the dividing point to column base is defined as the length of significant curvature localization zone \( L_{pc} \) in this work.

The influence of loading scheme on \( L_{pc} \) can also be found in Figure 5. It is clearly seen that the dividing point mentioned above moves away from the column base when the column is subjected to cyclic loading. In addition, the maximum curvature value increases from \( 6.67 \times 10^{-5} \) mm\(^{-1} \) to \( 7.74 \times 10^{-5} \) mm\(^{-1} \) when the lateral loading scheme changes from monotonic to cyclic. Both tensile strain and compressive strain contribute to curvature. Therefore, curvature concentration is caused not only by rebar yielding but also by concrete plastic deformation. Since both \( L_{sy} \) and \( L_{cs} \) of column under cyclic loading are larger than those under monotonic loading, as a result, \( L_{pc} \) are much larger for the column subjected to cyclic loading.
Figure 5 Curvature distribution of columns: (a) under monotonic loading and (b) under cyclic loading

Figure 6 Effect of length of FRP jacket on $L_{cs}$

Figure 7 Effect of confinement ratio on critical FRP length.

**MINIMUM JACKETING LENGTH FOR FLEXURAL RETROFITTING**

FRP was applied on the whole length of the columns in the above studies. However, it is not necessary for flexural retrofitting as FRP jacket is only needed in the plastic hinge zone. The length of concrete crushing zone should be known for design of retrofitting work. Figure 6 shows the effect of the length of FRP jacket on $L_{cs}$, where the FRP confinement ratio $\lambda$ is defined by

$$\lambda = \frac{f_i}{f_c} = \frac{2E_f e_{fu} f_i}{bf_c}$$

where $f_i$ is the confining pressure provided by FRP jacket; $b$ is the column depth. The length of FRP jacket is calculated from the column base. The dashed line is used to determine the minimum length of FRP jacket, referred to as the critical $L_{cs}$ in this work. A point on the dashed line means that FRP jacket length exactly equals $L_{cs}$. The points on the left side of the intersecting point between the dashed line and a curve mean insufficient FRP jacket length while the points on the right side indicate excessive jacket length. It can be seen from Figure 6 that $L_{cs}$ remains almost constant after the length of FRP jacket is larger than the critical $L_{cs}$. Considering that FRP jacket plays a role only when concrete significantly dilates or the axial strain of concrete exceeds 0.002, FRP jacket length needs to be only equal to the critical $L_{cs}$.

To validate the result, columns with full length jacketing are also shown in the figure. It is clear that the critical $L_{cs}$ is very close to $L_{cs}$ with full FRP jacketing and both decrease as confinement ratio increases.

**CONCLUSIONS**

This paper investigates the plastic hinge region of FRP-confined RC columns by the finite element method. The accuracy of the FEM model was first calibrated with test results. A systematic study was subsequently carried out to investigate the detail of physical plastic hinge regions, involving the rebar yielding zone, concrete crushing zone...
and curvature localization zone of FRP-confined RC columns. The plastic hinge lengths of FRP-confined RC columns under cyclic loading are larger than those of columns under monotonic loading. For flexural retrofitting of RC columns, FRP jacket needs only to cover the length of the concrete crushing zone, which is significantly affected by FRP confinement ratio.

ACKNOWLEDGEMENT

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REFERENCES

SEISMIC RETROFITTING OF RC WALLS EXTERNALLY STRENGTHENED BY FLAX-FRP STRIPS

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ABSTRACT

Several studies available in the literature demonstrate that natural fibers can be employed in Externally Bonded (EB) Fiber-Reinforced Polymer (FRP) strips for strengthening existing Reinforced Concrete (RC) members. The study aims at demonstrating the feasibility of retrofitting RC walls by using Fiber-Reinforced Polymers with Flax fibers (FFRP). The study consists of an experimental campaign and some analytical evaluations, the latter being intended at determining the evolution of relevant damage indices during the cyclic loading process. The behaviour observed in the tests on RC walls strengthened by FFRP are compared with both a reference wall and similar specimens strengthened with more conventional composite materials (i.e. Carbon-FRP, CFRP). The test results show that FFRP have the potential to be used for seismic retrofitting as a viable alternative to more common FRP materials and other traditional techniques. Indeed, RC wall specimens strengthened with FFRP show strength increases up to 150% and a ductility gains equal to about 30%. Moreover, the tests show that the walls strengthened by FFRP generally dissipate more energy than the ones strengthened with CFRP: this is an important property for seismic strengthening and retrofitting of existing RC structures.

KEYWORDS

FRP, RC walls, seismic strengthening, natural fibers, flax fibers.

INTRODUCTION

In recent years, seismic codes and guidelines have undergone a progressive evolution, driven by both earthquake engineering and materials technology. Since a couple of decades ago, reinforced concrete (RC) structures were simply designed for resistance (Antoniades et al. 2005; Brun 2002) and no attention was placed on displacement capacity and ductility under seismic actions. Therefore, existing structures do not meet the requirements of seismic capacity design and are generally in need for strengthening and retrofitting. Reinforced Polymer (FRP) strips, mainly made with either carbon (C) or glass (G) fibers, have demonstrated to be a viable retrofitting solution for increasing both force and displacement capacity. However, more innovative composites based on employing natural fibers, such as flax ones, are emerging as a promising alternative, featuring similar mechanical behaviour and enhanced sustainability properties.

This paper presents the experimental results obtained on RC walls strengthened with Flax-Fiber-Reinforced Polymer (FFRP) strips externally bonded (EB) to the concrete surface via epoxy resins. More specifically, it reports the results of the tests carried out on strengthened RC walls subjected to horizontal cyclic actions experimental results allow assessing the consequences of alternative arrangements of FRP strips applied on five of the tested specimens, also in comparison with the performance observed on the unstrengthened one assumed as a reference. This comparison is not only carried out in terms of cyclic and envelop force-displacement curves, but also by determining the evolution of well-known damage indices describing the energy dissipation capability under cyclic actions. Moreover, the results obtained on FFRP-strengthened walls are compared with the one obtained in similar tests carried out on RC walls externally strengthened by CFRP strips. Therefore, Section 2 describes specimen characteristics and preparation process, along with the adopted testing and measurement methods. Section 3 summarises the experimental observations and Section 4 proposes the aforementioned analyses in terms of damage.
EXPERIMENTAL DETAILS
Specimens and test description

Figure 1 depicts the main geometric properties and structural details of the tested RC walls: they are similar to the ones tested by Greifenhagen and Lestuzzi (2005) and are slender in shape, with a height-to-width ratio $H/L$ slightly higher than 2.5. Moreover, the chosen dimensions ideally correspond to a 1/3 scaled wall.

The internal steel reinforcement conforms to the minimal recommendations of Eurocode 2 (CEN, 2004). The overall dimensions of walls, steel reinforcement and the properties of materials (steel and concrete) are similar for all specimens. Hence, only the composite strengthening configuration differentiates the specimens (Table 1).

<table>
<thead>
<tr>
<th>Specimen label</th>
<th>Type of reinforcement</th>
<th>Nb layer</th>
<th>Disposition</th>
<th>Anchorage system</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Strip width</td>
<td>Wall-foundation</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Center</td>
<td>[number of wisp]</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>outside</td>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Transversal</td>
<td></td>
</tr>
<tr>
<td>SL3</td>
<td>No</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>SLR4</td>
<td>Bidirectional carbon</td>
<td>1</td>
<td>50</td>
<td>26</td>
</tr>
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<td></td>
<td></td>
<td></td>
<td>50</td>
<td>26</td>
</tr>
<tr>
<td>SLR6</td>
<td>Unidirectional carbon</td>
<td>1</td>
<td>50</td>
<td>44</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>75</td>
<td>60</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4</td>
</tr>
<tr>
<td>FRSL1</td>
<td>Unidirectional flax</td>
<td>3</td>
<td>-</td>
<td>4 × [44] Carbon</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>FRSL2</td>
<td>Unidirectional flax</td>
<td>4</td>
<td>-</td>
<td>4 × [44] Carbon</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>FRSL3</td>
<td>Unidirectional flax + Glass</td>
<td>2 + 2</td>
<td>-</td>
<td>100</td>
</tr>
</tbody>
</table>

The RC wall referred to as SL3 was not strengthened and, hence, it can be assumed as a reference. Conversely, the two walls SLR4 and SLR6 were strengthened by high-strength bidirectional strips of a wet lay-up CFRP system. The matrix used is epoxy type, cured at room temperature; moreover, CFRP is characterized by a modulus of elasticity equal to 105 GPa, and an ultimate strength of 820 MPa, determined for a nominal thickness of 1 mm. The other three walls (referred to as FRSL1, FRSL2 and FRSL3) were strengthened by unidirectional FFRP strips, whose modulus of elasticity is about 14.0 GPa, whereas the ultimate strength is around 120 MPa. It is worth highlighting that the FRSL3 also includes a bi-directional glass fabric, whose elastic modulus and ultimate strength are 9.0 GPa and 135 MPa, respectively. Concrete is nominally identical for all the walls: an average compressive
strength of 30 MPa was determined experimentally on samples of the materials constituting the RC walls. Figure 2 depicts the geometric layout of the various arrangements of FRP strips employed in the tested specimens.

Instrumentation and experimental protocol

The walls were tested under a constant vertical load of 90 kN, applied by an actuator, and a cyclic horizontal action, applied at a constant rate of 30 mm/min. The load procedure, taken from the literature (Inoue et al. 1997; Ile 2000) consisted of a sinusoidal load history whose amplitude grows of 1 mm every three cycles (Figure 3). As for measurements, 4 LVDTs were employed for monitoring the occurrence of possible undesired sliding or lifting of the wall specimen. Moreover, 2 LDVTs were placed along the diagonals to verify the length changes under tension or compression during the application of the cyclic action. Finally, some strain gauges were bonded on both internal steel reinforcement and externally bonded FRP strips.
RESULTS

Figure 4 reports the cyclic load-displacement curves obtained in some of the experimental tests: the two graphics document the hysteretic curves of specimens SL3 and SLR6 and, hence, it allows a first qualitative comparison between the reference specimen (SL3) and one of the CFRP-strengthened walls (SLR6).

Therefore, Figure 4 allows understanding the effect of CFRP strengthening and the corresponding enhancement the original cyclic response obtained on the reference specimen. Figure 5 highlights the difference among the responses of all tested specimens in terms of envelop curves.

The following Table 2 summarizes the main relevant quantities measured in the six experimental tests. They clearly show the effectiveness of the composite reinforcement, as load capacity increases by at least 30%. FRPR strengthening is even more effective, as the increase in resistance is about 100% (FRSL1/2) or even 140% (FRSL3). Moreover, displacement capacity is also influenced by FRP strengthening. More specifically, two kinds of behavior were observed: on the one hand, RC walls strengthened by CFRP strips collapse at displacement levels lower than the reference specimen; conversely, on the other hand, an increase of up to 30% (FRSL3) was observed for FFRP-strengthened walls. Therefore, under the seismic point of view, the FRP configuration adopted in specimen FRSL3 has a great potential for applications in seismic retrofitting: in fact, the GFRP sheet placed on the lateral face of the RC wall (Figure 2) makes it possible to bridge cracks opening in concrete between the longitudinal FFRP strips.

<table>
<thead>
<tr>
<th>Name</th>
<th>$F_u^-$ [kN]</th>
<th>[%]</th>
<th>$\delta_u^-$ [mm]</th>
<th>[%]</th>
<th>$F_u^+$ [kN]</th>
<th>[%]</th>
<th>$\delta_u^+$ [mm]</th>
<th>[%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>SL3</td>
<td>27.75</td>
<td>-</td>
<td>20.56</td>
<td>-</td>
<td>27.43</td>
<td>-</td>
<td>19.91</td>
<td>-</td>
</tr>
<tr>
<td>SLR4</td>
<td>36.01 +30</td>
<td>20.52</td>
<td>1</td>
<td>42.25 +54</td>
<td>21.08 +6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SLR6</td>
<td>47.24 +70</td>
<td>14.57</td>
<td>30</td>
<td>52.23 +90</td>
<td>13.47 -27</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FRSL1</td>
<td>55.25 +99</td>
<td>18.82</td>
<td>8</td>
<td>50.37 +84</td>
<td>22.40 +13</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FRSL2</td>
<td>54.00 +94</td>
<td>22.92</td>
<td>11</td>
<td>45.06 +64</td>
<td>21.78 +9</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FRSL3</td>
<td>68.50 +147</td>
<td>26.86</td>
<td>+31</td>
<td>56.75 +107</td>
<td>23.11 +16</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$X_u^-$: maximal value in push, $X_u^+$: maximal value in pull

Figure 6 Failure observed during test
Finally, CFRP strips failed due to either peeling from concrete (SLR6) or pulling of external anchorage (SLR4). The strain measurements on CFRP show a effectiveness of about 30%. As for FFRP strengthening, recorded deformations show full utilization of composite, resulting in tension failure (red circle in Figure 6).

ANALYSIS AN DISCUSSION

Stiffness analysis

Stiffness degradation can be observed in the experimental tests as the maximum displacement grows up and the number of cycles increases (Figure 4). This is witnessed by a progressive reduction in slope of the load-displacement response both in the reloading and unloading phases. In fact, the mechanical properties of materials tend to decrease, as damage develops within the tested specimen. For instance, cracking of concrete causes a loss in the monolithic structure of this material, which may also be interpreted as a reduction of the effective flexural stiffness of the cross section.

The evolution of stiffness during the cyclic load protocol was quantified by determining the average stiffness for each group of three equal cycles imposed for each displacement amplitude (Figure 3). Figure 7 shows these stiffness evolutions observed in the three walls strengthened by FFRP. Stiffness drops very quickly for both the reference walls and the three walls reinforced by FFRP. However, at the maximum displacement recorded for the unreinforced wall, the residual stiffness of FFRP strengthened walls is about three times the value determined for the reference specimen.

![Figure 7](image1.png) Evolution of stiffness in walls: reference and FFRP strengthened RC walls

![Figure 8](image2.png) Evolution of cumulative elastic energy: reference and FFRP strengthened RC walls.
Dissipated energy analysis

For each specimen, elastic and dissipated energies can be defined by processing the hysteresis curves recorded during the experimental test (Dazio et al. 1999). Figure 8 reports the evolution of cumulative elastic energy and shows that FFRP strengthened specimens (and, particularly, FRSL3) result in significantly higher values of this quantity, which, in order word, means a lower damage.

The comparison between the performance of CFRP- and FFRP-strengthened RC walls, already proposed in terms of force-displacement curves, may be further analyzed in terms of dissipated energy during the cyclic load process. Figure 9 shows that FFRP strengthened walls (FRSL3) are fairly more efficient in terms of dissipated energy (also in cumulative terms) than the corresponding CFRP strengthened specimens (SLR4).

![Graphs showing dissipated energy and cumulative elastic energy](image)

Figure 9 Evolution of energy of walls reinforced by flax fibers FSLR3 and carbon SLR4

CONCLUSION

This experimental study investigated the cyclic response of RC walls externally strengthened by FRP strips with the main aim to compare the performance of more conventional carbon-based solutions, with more innovative ones based on using natural fibers made of flax. Besides the clear enhancement in sustainability, FFRP demonstrated several advantage in terms of mechanical performance. They may be summarized in the following:

- FFRP have shown increases up to 150% in resistance, which are similar to the effects observed on CFRP-strengthened RC walls;
- moreover, the former often led to an increase in displacement capacity (close to 30% for the specimen FRSL3, in which FFRP and GFRP were employed jointly); conversely, the response of CFRP-strengthened RC walls was generally less ductile than the one observed for the reference specimen;
- furthermore, the cyclic response of FFRP-strengthened walls was less affected by the progressive degradation in stiffness and energy dissipation that is typical in existing RC structures;
- therefore, RC walls externally strengthened by FFRP (and especially the specimen FRSL3) have a more dissipative response, which is an important property for seismic retrofitting.

Finally, these results demonstrate that FFRP is a competitive alternative to more conventional solutions based on using C- or G-fibers and, hence, further efforts are justified for advancing knowledge on this materials and their interaction with existing materials and members, toward the formulation of sound seismic design formulae.

REFERENCES


STRENGTHENING OF RC SHEAR WALLS WITH CFRP STRIPS

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2Technology Faculty, Civil Engineering Department, Gazi University, Turkey
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ABSTRACT

The purpose of this study was to investigate the hysteretic behavior of shear deficient reinforced concrete (RC) walls that were strengthened with carbon fiber reinforced polymer (CFRP) strips. Totally, ½ scale five specimens with 1.5 aspect ratio walls were constructed. One of them was tested without any retrofitting as a reference specimen and four of them were retrofitted specimens with CFRP strips. All of the specimens were tested under cyclic lateral loading. CFRP strips with different configurations were tested like X-shaped, horizontal and parallel strips or combinations of them. All of the CFRP configurations were symmetrically bonded to both sides of the shear wall and were anchoraged to the wall. The research focuses on the effect of using CFRP strips for enhancing strength and increasing ductility of the non-seismic detailed shear walls. Test results shows that all of the CFRP strip configurations significantly improves the lateral strength, energy dissipation and deformation capacity of the shear deficient RC walls. The specimen that was strengthened with X-shaped CFRP strips was failed with premature shear failure. The specimen that was strengthened with horizontal strips was showed flexural hysteretic behavior and plastic hinge was developed at the wall base. CFRP strips were controlled shear crack propagation and resulted in improvement of displacement capacity.

KEYWORDS

CFRP, RC shear wall, strengthening, cyclic load.

INTRODUCTION

In earthquake resistant design, shear walls are common lateral load resisting systems found in many reinforced concrete (RC) structures. Limited numbers of RC shear walls are in use at older buildings. A number of buildings were found with walls having cross sections with relatively small aspect ratios \( h/l \), \( h \) = wall height, \( l \) = wall length, which in some cases resembled like elongated rectangular columns. These buildings survive from the earthquake with minor damages at structural framing system, but major damages at the masonry walls. Poorly designed and detailed shear walls in these buildings that are damaged with diagonal shear cracks, survived from the earthquake, and saved the structure. The shear walls of the numerous existing buildings have number of design and construction detailing deficiencies such as poor or no confinement of boundary element with enough reinforcement, poor or no bonding of the transverse reinforcement to concrete, and inadequate shear strength for preventing development of hinging. Consequently, strengthening for improving the shear capacity of the RC walls became more of a concern in the area of seismic design for RC structures (Saatcioglu et al., 2001). Researches on the strengthening of RC walls by FRP sheets are few in number. One of the firstly performed experimental study on the strengthening of RC walls is the application of FRP sheets with fibers to the wall sides in the vertical direction against the cyclic shear and flexure (Lombard et al., 2000), and the other one is the application of different configurations of externally bonded FRP reinforcement to wall like columns under uni-axial compression (Lombard et al., 2000; Neale et al., 1997). There are some other studies like the application of wing walls to RC columns and the application of unreinforced concrete infill walls (Iso et al., 2000; Sugiyama et al., 2000). Even though the strengthening of shear deficient reinforced concrete walls using FRP strips is an easily applicable and practical technique; there is a lack of information in the literature about the subject such as strip layout can be more effective on the hysteretic behavior. The main focus of the present research is the experimental analysis of the proper CFRP strip configurations for improving the hysteretic behavior of shear deficient reinforced walls under lateral loading. The goal of the retrofitting is to improve the shear strength, ductility and energy dissipation of the poor detailed RC walls. In the study, shear deficient RC walls are strengthened by four different configurations of CFRP strips and are tested under cyclic lateral loading. The different configurations of CFRP strips are cited as follows; the
lateral strips, X-shaped strips, combination of X-shaped and lateral strip, and combination X-shaped and parallel strips (Altun et al., 2013).

EXPERIMENTAL STUDY

Five shear deficient, $\frac{1}{2}$ scaled RC wall specimens were constructed and tested at the laboratory for investigating the influence of shear strengthening that were achieved by using four different CFRP configurations on the hysteretic response of structural walls. Dimensions and reinforcement details of the test specimens are given at Figure 1. Wall length, height and thickness are $l=1000$ mm, $h=1500$ mm, and $t=100$ mm, respectively. The aspect ratio of the wall ($h/l$) is 1.5. Vertical and lateral reinforcement ratios of the wall are $\rho_v=0.0183$ and $\rho_h=0.0015$, respectively. In addition, each of the lateral reinforcement in the wall is anchored by 90 degree hooks at the ends of the wall and there are no boundary members at the sides of the walls to simulate the poor details in the existing structures.

Specimen 1 was the reference specimen that was tested without strengthening. The other four specimens (Specimens 2, 3, 4 and 5) were tested after strengthening with four different configurations of CFRP strips. CFRP strips were symmetrically applied to both sides of the concrete wall. The detailed descriptions of applied CFRP configurations are given in Figure 2. All of the CFRP strips were anchoraged to the concrete wall by fan type of anchorages. The distances between the anchorages were 300 mm and 270 mm for lateral and diagonal strips, respectively. The photographs of fan type anchorages are presented in Figure 2. In the experimental study, specimens with low compressive strength were constructed to represent the concrete strength of the existing old buildings. The concrete strength of the test wall was approximately 15 MPa on the day of testing. Properties of reinforcement, CFRP and resin that are used in specimen are given in Table 1 and Table 2, respectively.

The test setup, loading system, and instrumentation are shown in Figure 3. Specimens were tested under cyclic lateral loading. Each specimen was loaded laterally as a vertical cantilever with forces applied through the top beam. No axial load was applied to specimens. In the instrumentation of the specimens, strain gauge based linear variable differential transformers (LVDTs) were used for displacement measurements. Additionally, strains in the CFRP strips were measured using strain gauge. Strain gauge locations were given in Figure 2.

Table 1. Properties of Reinforcements

<table>
<thead>
<tr>
<th>Reinforcement Diameter (mm)</th>
<th>Yield Strength $f_{ys}$ (MPa)</th>
<th>Failure Strength $f_{ys}$ (MPa)</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>325</td>
<td>420</td>
<td>Plain</td>
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<td>10</td>
<td>430</td>
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<tr>
<td>12</td>
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<tr>
<td>16</td>
<td>425</td>
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<td>Properties of CFRP</td>
<td>Remarks of CFRP</td>
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<td></td>
</tr>
<tr>
<td>--------------------</td>
<td>-----------------</td>
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</tr>
<tr>
<td>Thickness (mm)</td>
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<tr>
<td>Tensile Strength (MPa)</td>
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<table>
<thead>
<tr>
<th>Properties of Resin</th>
<th>Remarks of Resin</th>
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<td>30</td>
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<tr>
<td>Elastic Modulus (MPa)</td>
<td>3800</td>
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</table>

Figure 2 Strengthening Schemes of Specimens and Anchorage Details (Dimensions in mm.)

Figure 3 Test Setup and Instrumentation of Specimen (Dimensions in mm.)
EXPERIMENTAL RESULTS

Response envelopes for specimens shown in Figure 4 are plotted by connecting the peak points of the base shear-top story displacement hysteretic curves for each specimen. As can be seen from this figure lateral displacement, lateral load carrying and energy dissipation capacities were significantly increased. Fan anchorages were prevented the premature debonding of CFRP strips from the wall surfaces. Hence, CFRP strips were controlled the widening of the shear cracks and lateral displacement significantly increased. Shear sliding deformation was not observed during the tests. Test results are summarized in Table 3. Initial stiffness was defined as the initial slope of load displacement curve of the first forward half cycle. The stiffness at the ultimate load was defined as the average of the slopes of linear lines connecting the ultimate loads with the origin of load displacement curves in the forward and backward half cycles. The ductility ratios of the specimens were calculated by using the ratio of lateral displacement that are measured at the first yielding of vertical reinforcements of wall side to the displacement that are measured at the point where the ultimate flexural shear capacity decreased to 85%. The area under the hysteresis loops can be used as a measure of the energy dissipation capacities. Energy dissipations capacities were determined by calculating the areas inside the hysteretic load-displacement curves for each of the specimens. The cumulative energy dissipation capacities were defined as the sum of the area enclosed by all hysteresis loops.

The photographs of specimen after failure are presented in Figure 5. Reference specimen 1 was failed due to premature shear failure due to concentrated shear cracks along both of the diagonals of the wall. Specimen 2 was reached its flexural capacity and showed a ductile flexural behavior. Vertical reinforcements on the wall sides of Specimen 2 were yielded when lateral displacements reached to 0.78% and 0.86% drifts during forward and backward loading, respectively. Plastic hinge was developed at the base of the wall. Concrete cover was crushed over the base under compression forces. Specimen 3 was failed due to shear before reaching its flexural capacity. Shear cracks were concentrated through the diagonals of the wall and these cracks were reached to the wall corners with the increasing lateral load. The concrete cover was crushed at the bottom corners of the wall. The concrete was crushed at the bottom corners of the wall under the compressive stresses caused fan anchorages to be damaged. Specimen was failed due to premature shear.

Table 3. Experimental Results

<table>
<thead>
<tr>
<th>Spec. No</th>
<th>Ultimate Load (kN)</th>
<th>Drift Ratio (%)</th>
<th>Ductility Ratio</th>
<th>Stiffness (kN/mm)</th>
<th>Failure Mechanism</th>
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<tr>
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<td>5</td>
<td>249</td>
<td>0.82</td>
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Specimen 4 was reached its flexural capacity. The vertical reinforcement at the wall sides were yielded when 0.82% and 0.90% lateral drift ratios were reached during forward and backward loading, respectively. Diagonal CFRP strips were debonded from the wall surface between the anchorages with the increased lateral drift cycles. Shear cracks were concentrated through the diagonals of the wall. Specimen 4 was failed with shear failure. Specimen 5 was reached its flexural capacity. The vertical reinforcement at the wall sides were yielded when 0.82% and 0.84% lateral drift ratios were reached in the forward and backward cycles, respectively. The CFRP strips were ruptured at the lateral drift ratio of 2.5%, and the lateral load carrying capacity suddenly decreased. Specimen was collapsed with the shear failure.

Lateral strengths of all strengthened specimens were significantly increased compared to the reference specimen. Ultimate flexural shear strengths of the strengthened specimens were 1.54-1.67 times greater than that of the reference specimen. The strengthened specimens 2, 4 and 5 were reached the flexural capacity with almost the same lateral stiffness and were showed a ductile behavior. A specimen 3 displacement capacity was remained restricted due to premature shear failure. After reaching the 18% (on the average) of the nominal flexural strength, the lateral stiffness was decreased at the specimens. However, CFRP strips were restricted the widening of shear cracks on the wall.

![Specimens after Failure](image)

Even though the similar strengthening configurations were used at Specimen 3 and Specimen 4, they were showed completely different hysteretic lateral behavior than each other. Specimen 4’s wall lateral displacement was significantly increased due to the increase in concrete ultimate strain at the lower and upper parts of the wall under compression forces. The initial stiffness of the reference specimen was 47.6 kN/mm. The initial stiffnesses that were measured for the strengthened specimens were obtained slightly greater than the reference specimen. The average of strengthened specimens’ initial stiffnesses was 56.7 kN/mm. Shear cracks were caused significant drop at the lateral stiffnesses. The average of measured lateral stiffnesses of the specimens was 17.9 kN/mm at the ultimate load. At the ultimate load, the averages of initial stiffnesses were decreased 106%. Specimen 1 and 3 were showed non-ductile, Specimen 2, 4 and 5 were showed ductile hysteretic behaviors. Ductility ratios were measured as 3.47, 2.70 and 3.05 for the Specimen 2, 4 and 5, respectively. Energy dissipation of the strengthened specimens was significantly larger than that of the reference specimen. The ratio of energy dissipation capacities of strengthened specimen to that of reference specimen changes in between 2.69–5.15. Among the strengthened specimens, the lowest energy dissipation capacity belongs to Specimen 3, and largest energy dissipation capacity
belongs to Specimen 2. Specimen 4 and 5 consumes almost the same amount of energy. The ratio of energy dissipation capacity of Specimen 4 and 5 to that of the reference specimen is 4.34 on the average.

The typical examples of strain measurements taken from the CFRP strips during the tests are given in Figure 6. The mean of the maximum strains that were measured from lateral strips was 0.0090 mm/mm for the Specimen 2. The measured maximum strains from the diagonal CFRP strips were 0.00748 mm/mm and 0.00840 mm/mm for the Specimen 3 and Specimen 4, respectively. The measured maximum strain was 0.00813 mm/mm for the Specimen 5. Deformations and shear cracks that were developed on the wall were caused debonding of CFRP from the wall surface. The debonded CFRP fibers were remained under the influence of combined stresses along with the tensile stresses and therefore this combined stress were caused fiber ruptures before reaching the ultimate strain.

CONCLUSIONS

In this study, the hysteretic behaviors of strengthened shear deficient reinforced concrete walls by CFRP strips under the lateral loading are experimentally investigated. From this study, the followings were concluded:

1. The strengthening of shear deficient reinforced concrete walls by using CFRP strips was an effective technique. Usage of CFRP strips significantly improves the hysteretic behavior of shear deficient reinforced concrete walls under the cyclic lateral loading. The strip configurations were effective on the hysteretic behavior of strengthened wall and failure mode. The best performance for the improvement of lateral displacement capacity and lateral strength of shear deficient RC walls has been obtained from the strengthening with lateral strips. The wall strengthened by lateral strips is showed a ductile flexural hysteretic behavior and at the base of the wall plastic hinge has been developed. Although the X-shaped strips is increased the wall shear strength, they are not enough to reach the flexural capacity. In the walls strengthened by X-shaped and lateral strip combination, and X-shaped parallel strips, flexural capacity has been reached.

2. CFRP strips are not effective on improving the initial lateral stiffness of the specimens. However, restriction of the widening of shear cracks are developed on the wall by the CFRP strips are controlled the decrease in the lateral stiffness. Except the Specimen 3, all other strengthened specimens are reached flexural capacity with almost the same lateral stiffness.

3. Specimens that are strengthened by lateral and X-shaped CFRP strips (Specimen 2 and 5), a significant decrease in the flexural strength of the specimen has not been observed till 2.0% lateral drift. The displacement capacities of the specimens have been improved by the applied strengthening technique without causing any significant decrease in their load carrying capacities.

4. The strengthened specimens are dissipated much more energy than the reference specimen. The ratio of energy dissipation of strengthened specimens to that of the reference specimen has been changing in between 2.69 and 5.15.

5. Fan anchorages are prevented the debonding of the CFRP strips from the reinforced concrete wall. As a result of prevention of debonding of CFRP strips totally by the anchorages, the tensile forces of the strips are provided the load carrying capacities to continue till reaching considerable lateral displacements. The maximum strains are measured on CFRP strips were 0.008-0.009 mm/mm.
ACKNOWLEDGMENTS

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Mini-symposium on Prestressed FRP for Strengthening and New Constructions

Organizers:
Raafat EL-HACHA
Hothifa ROJOB
FRP PRESTRESSING SYSTEMS FOR FLEXURAL STRENGTHENING OF STRUCTURAL ELEMENTS – A REVIEW

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ABSTRACT

The application of Fibre Reinforced Polymers (FRP) in strengthening and retrofitting of existing structures has emerged as a practical and efficient technique compared to the application of conventional materials (i.e. steel and concrete). In the case of flexural strengthening of beams, the FRP strengthening can be categorized into two main systems, the passive and the active system. The passive system doesn’t involve any prestressing to the FRP material; consequently, the passive system has minimal impact on the performance of the beam at the service conditions. On the other hand, the active strengthening involves the application of prestressing force to the FRP material, which consequently improves the beam performance at the service conditions. The challenging part of the active system is the application of the prestressing force to the FRP material. This paper presents the developments in the area of prestressing methods of FRP materials for flexural strengthening of steel and concrete structural members (beams and slabs). Several types of prestressing systems are presented with the focus on the prestressing systems where the FRP is prestressed against the member itself. The systems include prestressing of FRP materials, either as externally bonded sheets and strips at the surface of the RC beams or slabs and steel beams or as Near-Surface Mounted (NSM) strips and bars in the case of RC beams.

KEYWORDS

Active, passive, FRP, prestressing, NSM, anchorage, sheets, strips, gradient.

BACKGROUND

The high strength-to-weight ratio, corrosion resistivity and easiness of handling, are the main advantages of the FRP material compared to the conventional structural material, which made the FRP as the material of choice when it comes to strengthening and rehabilitation of concrete and steel structures. In the early applications of FRP strengthening, the FRP sheets/ strips were epoxy bonded to the tension face of concrete or steel beams and concrete slabs in a system known as Externally Bonded (EB). Later, the Near-Surface Mounted (NSM) system was introduced, where the FRP strips/bars are embedded inside a pre-cut groove in the tension face of the concrete beam filled with epoxy adhesive. Other systems have also been implemented such as the Mechanically Fastened (MF) FRP laminates or hybrid systems of MF and EB plates. Schematic of these system is shown in Figure 1. Most of the FRP applications were passive strengthening, where no prestressing force is applied to the material. In the passive system, the ultimate capacity of the strengthened beam is improved, however, it has minimal effect on the flexural behaviour at service loads. In other words, when the non-prestressed FRP is applied to the beam, the FRP will not be activated unless further deflection/load is induced to the beam (El-Hacha and Soudki 2013). In contrast, the active strengthening system with prestressed FRP enhances the flexural performance of the beam at service conditions by reducing deflection and cracking-in concrete members.–
Figure 2 shows the performance of RC beams strengthened with prestressed and non-prestressed FRP. The current paper presents a brief literature review on the mechanisms used to apply the prestressing force in the FRP material for flexural strengthening of structural steel and concrete members. More focus will be given to RC beams and slabs as most of the research on prestressed FRP was in this area.

**PRESTRESSING METHODS**

**Indirect prestressing method**

The methods used to apply the prestressing force to the FRP have evolved in the last two decades. In the early stages, researchers used an indirect system to induce the prestressing forces to the FRP material (El-Hacha and Soudki 2013). The system was first introduced at EMPA in 1992 (Meier et al. 1992, and Deuring 1993). Figure 3 shows a top view schematic diagram of the indirect prestressing system. The system involves inducing the required prestressing force to the FRP by tensioning the FRP against an external reaction frame independent of the beam until the epoxy cures. After the epoxy gains its strength, the prestressing system is released and the prestressing force is transferred to the concrete beam (Nordin et al. 2001, Nordin 2003, Casadei et al. 2006, Nordin and Täljsten 2006, Jung et al. 2007, and Wu et al. 2007). Yu et al. (2004) prestressed the FRP sheets against a steel beam before bonding the sheets to the concrete beam as shown in Figure 4. The indirect system has been criticized for the lack of practicality and the requirements for special tools and equipment.

![Figure 3 Schematic of the indirect prestressing systems (top view).](image)

**Direct prestressing method**

The direct system involves prestressing the FRP material against the beam itself directly without the need for external reaction frame. Unlike the indirect method, this system gained more acceptance in the research community and was superior over the indirect system for being more practical. Figure 5 illustrates the main features of this system. The system typically consists of a dead end, where the FRP material is connected to the concrete using anchorage plate, while the other end is the live end, where the FRP is pulled by a certain mechanism (mainly hydraulic jacks). It should be noted that recently El-Hacha and Hadiseraji (2014) substituted the hydraulic jacks with Shape Memory Alloy bars that worked as actuators. The FRP is prestressed as shown in Figure 5 and once the desired load is attained, the live end will be anchored to the concrete. Once the live end is anchored, the supplementary tools (the bearing plates and the loading tool) can be removed (El-Hacha et al. 2001, and Wu 2007). The FRP material in this system can be either NSM FRP strips/bars or EB-FRP plates/sheets.

![Figure 5 Schematic of the direct prestressing systems (top view).](image)
The earlier applications of the direct prestressing were used for prestressing EB-FRP laminates/sheets. Wight et al. (1996) and Wight (1998) used the direct system to strengthening 5 m long reinforced and prestressed concrete beams. The system was too laborious, as shown in Figure 6 consisted of round bar anchors that gripped the CFRP sheets that were then attached to anchors bolted to the sides of the beams.

Similar systems used by Izumo et al. (1997) and Saeki et al. (1997) as shown in Figure 7. Darby et al. (1999) developed a prestressing system for the strengthening of cast iron beams of the Hythe Bridge in Oxford. The prestressing system consisted of FRP plates connected to the end anchors which are clamped to the beams (without the need for drilling) as shown in Figure 8. This was the first steel structure to be strengthened with prestressed FRP sheets. El-Hacha et al. (1999, 2000 and 2001) developed an innovative system that consisted of a movable flat steel plate anchor bonded to the sheets at the jacking end and another fixed steel angle anchor bonded to the sheets at the dead end and mounted on the concrete beam permanent anchor. The FRP sheet was stressed using a simple hydraulic jacking assembly causing the sheets to be directly prestressed against the beam itself as shown in Figure 9. The system was slightly modified for strengthening pre-damaged continuous RC T-section as shown in Figure 10. Other modifications were made for the purpose of strengthening of one-way RC slabs (Wight and Erki 2001, and El-Hacha et al. 2003) as shown in Figure 20, and two-way RC slabs shown in Figure 12 (Longworth et al. 2004). Honorio et al. (2002) used the system in a field application at the Canadian Forces Base in Kingston, Ontario, to strengthen a prototype bridge constructed from 18.0m long prestressed RC girders as shown in Figure 13. The system was also used successfully in 2003, and for the first time in North America, for strengthening a highway bridge in Winnipeg, Canada, as shown in Figure 14. Another application using a similar system was carried out in the late 1990s (Garden and Mays 1999, Darby et al. 2000, Fallis et al. 2004, and Ford 2004).

Kim et al. (2004) proposed a steel anchor plates for prestressing multi-layered FRP sheets. (Franca et al. 2007) used a similar system on the both sides of T-section webs to avoid drilling the bottom side of the beam because of the narrow web width. Franca (2007) modified his system to have the FRP at the bottom face of the beam but still with using side plates without drilling in the bottom face of the beam as shown in Figure 15.
Figure 9 The prestressing system developed by El-Hacha et al. 2000 and 2001

Figure 10 Modified prestressing system by El-Hacha, Wight et al. (2003) and Wight et al. (2001, 2003)

Figure 11 The prestressing system for one-way slabs by Wight and Erki 2001, El-Hacha, Wight et al. 2003

Figure 12 The prestressing system for two-way slabs by Longworth, Bizindavyi et al. (2004)

Figure 13 Strengthening of RC bridge girder in Kingston, Ontario (Honorio, Wight et al. 2002)

Figure 14 Strengthening of RC bridge girder using prestressed FRP sheets (Fallis, Eden et al. 2004)

Figure 15 The system developed by Franca (2007)
Many researches were carried out in the early 2000s on EB prestressed FRP plates/sheets which focused on the failure mechanisms, debonding issues, and other structural issue but there was no significant improvement on the prestressing methods. However, the development diverted to the NSM technique which started with the work of Al-Mayah et al. (2005) who used the direct system by tensioning the NSM CFRP bars against the ends of the beam. A hydraulic jack was used to tension the FRP bar against the steel wedge anchor. The system was used by other researchers with minor modifications (Badawi and Soudki 2006, Badawi 2007, Badawi and Soudki 2009). Figure 16 shows the general features of this system. This system required an access to the ends of the beams which is not practical in many cases. De-Lorenzis et al. (2002) suggested a direct system that can be used without needing the access to the sides or ends of the beam. The system consisted of threaded steel bar connected to the end of the FRP rod through a tube as shown in Figure 17. The force is applied to the FRP rod by manually rotating the steel nut. The drawback of this system is that the manually induced force (by tightening the nut), which is limited to lower values of prestressing forces. A breakthrough in the prestressing system was the work of Gaafar (2007). Figure 18 shows a schematic drawing of his system. The system was successfully used for flexural strengthening of the large-scale RC beam using NSM FRP bars and strips (El-Hacha and Gaafar 2011, Oudah 2011, Omran and El-Hacha 2014). The system was modified by El-Hacha and Hadiseraji (2014) by replacing the hydraulic jack with Shape Memory Alloy (SMA) bars as shown in Figure 19. The pre-strained SMA bars acted as actuators and the prestressing force was applied by activating the bars through heating. The same system was further modified by Hadiseraji and El-Hacha (2014) to apply a prestressing force to FRP sheets as shown in Figure 20.

**Modification to the direct system (Gradient anchorage method)**

The direct prestressing system -introduced earlier- requires end anchors to transfer the prestressing force to the concrete and to avoid premature peel-off failure. To avoid the use of anchors, while preventing the peel-off failure, the gradient anchorage method was developed at Empa (Stocklin and Meier 2001, Meier and Stocklin 2005, Motavalli et al. 2011). The system consists of two main units; the jacking unit that provides the desired prestressing force and a heating unit in front of it that accelerate the curing process of the epoxy resin. The system involves curing the resin before releasing the prestressing force. A mobile device cures the adhesive from the center of the laminate towards its ends. In such a case there is a control over the prestressing force over the length of the laminate,
and the amount of prestressing force is reduced towards the end of the prestressed sheet to reduce the risk of debonding as presented in Figure 21. The system was successfully used by Czadzierski and Motavalli (2007) for strengthening an RC bridge girder. The system was also used to strengthen RC slabs (Kotynia et al. 2011). Recently, the system was further improved by Michels et al. (2012 and 2013) to suit field applications as shown in Figure 22. Similarly, Monti and Liotta (2006) developed a system where the FRP is wrapped around a cylinder and the cylinder is mounted on a sliding car (Figure 23) that can move along the beam length and reduce the prestressing force towards the ends of the beam to eliminate the end debonding.

**Figure 21** Prestressing system developed by Meier and Stöcklin (2005).

**Figure 22** The system modified by Michels, Cruz et al. (2013)

**Figure 23** The system developed by Monti and Liotta (2006)

**Prestressing of steel girders (steel-concrete composites)**

Most of the FRP applications were used for strengthening RC structures and less work was done for steel structures. This is mainly attributed to the fact that the stiffness of the FRP material is relatively smaller than the steel; consequently, the FRP contribution in the flexural capacity of steel structures is relatively less than that for the less-stiff RC structures, unless high modulus FRP material is used to strengthen steel members. Nevertheless, prestressed FRP has been implemented in strengthening steel-concrete composite beams (Ragab et al. 2007). The first attempt to prestressed steel structure was done by Darby, Skwarski et al. (1999) as reported in the previous section. Prestressed CFRP strips were used to repair notched steel beam (Colombi et al. 2003, Zhou et al. 2013). Wipf et al. (2003) used post-tensioned CFRP rods mounted on the sides of the web of steel bridge girder. The CFRP rods we prestressed using a hydraulic jack bearing against a steel bracket bolted to the web of the beam (Figure 24). For flexural strengthening of steel-concrete composite beam Schncherch et al. (2005) used CFRP strips prestressed by the means of tightening threaded rods as shown in Figure 25. The fixed end consists of a tapered steel plate that is bolted to the beam and bonded to the CFRP while the Moveable end uses application of the load by fine-threaded bolts to prestress the CFRP. An anchoring system developed by Aly (2007) and El-Hacha and Aly (2013) was used to strengthen steel-concrete composite girders as shown in Figure 26. The system consisted of the fixed end anchor where a steel plate bonded to the FRP sheet is permanently bolted to the bottom flange of the beam, and movable end where a hydraulic jack was used to apply the prestressing force to the sheet.

**Figure 24** : Strengthening steel bridge girders with external post-tensioned CFRP rods (Wipf et al. (2003))

**Figure 25** The prestressing system developed by Schnerch (2007)

**Figure 26** Prestressing system developed by El-Hacha and Aly (2013)
CONCLUSIONS

This paper has presented a brief literature review on the development of prestressing systems for FRP flexural strengthening of beams and slabs. Most of the FRP prestressing applications were conducted on RC members and less work was done on steel members. The prestressing systems evolved from indirect system - where an external frame is used to prestressed the FRP before attaching it to the member - to a more robust direct prestressing system against the member itself, and the recent development of the gradient anchorage system which eliminates the need for end anchors. The NSM prestressed FRP become more favorable over the externally bonded prestressed FRP as it provides more protection to the FRP material and reduces the chances of debonding.

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TWO EXAMPLES OF POST-TENSIONED CFRP CABLES IN BRIDGE CONSTRUCTION: ONE IN REHABILITATION, ONE IN NEW CONSTRUCTION

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ABSTRACT

Since the early 1980ties it was the vision of Empa researchers to develop CFRP tendons for suspended bridges (Meier 1987) and for post-tensioning. The key issue was the development of a reliable anchorage system for CFRP parallel wire bundles. In 1996 there was a first real application with such an anchorage system for the CFRP stays at Stork Bridge in Winterthur. Two years later followed CFRP post-tensioning cables on the Kleine Emme Bridge as new construction and on the Verdasio Bridge as a rehabilitation project. The bicycle and pedestrian single span bridge over the River Kleine Emme near Lucerne was built in October 1998. The bridge deck is 3.8 m wide, 47 m long and has been designed for the maximum load of emergency vehicles. The superstructure is a space truss of steel pipes in composite action with the steel rebar reinforced concrete deck. The bottom chord, a tube of 355-mm diameter, was post-tensioned with two CFRP cables inside the tube. Each cable was built up with 91 pultruded CFRP wires of 5-mm diameter. The post-tensioning force of each cable is 2.4 MN. Therefore the CFRP wires are loaded with a sustained stress of 1350 MPa. The Verdasio Bridge in the south of Switzerland is a two-lane highway bridge and was built in the seventies. The length of the continuous two-span girder is 69 m. A large internal post-tensioning steel cable positioned in a concrete web was fully corroded. It had to be replaced in December 1998. Four external CFRP cables arranged in a polygonal layout at the inner face of the affected web inside of the box replaced the corroded steel cross section. Each cable was made up of 19 pultruded CFRP wires with a diameter of 5 mm. This project is as far most interesting as in this case the sustained post-tensioning stress on the cable cross sections is in average as high as 1806 MPa. The results of the long-term monitoring are for both examples excellent and prove especially the high reliability of the CFRP anchorage system.

KEYWORDS
CFRP, tendons, strengthening, post-tensioning, anchorage system, stress relaxation.

INTRODUCTION

World’s first application of carbon fibre reinforced polymers (CFRPs) in construction happened at Ibach Bridge near Lucerne in 1991 (Meier 1995). At that time, Empa full scale laboratory experiments proved already the feasibility of the application of pre-tensioned CFRP strips for post-strengthening (Deuring 1993). Pre-tensioned CFRP strips make much better use of the high strength of the expensive material. However the responsible building authorities estimated the risk of a failure as too high. Therefore “only” not pre-tensioned, loose CFRP strips had been applied. This application still performs perfectly well after a quarter of a century. Meanwhile pre-stressed CFRP strips have also been accepted in practice (e.g. Motavalli et al. 2011, Kotynia et al. 2015, Ghafoori et al. 2015). Beside pre-stressed CFRP strips Empa research has been focused since the early 1980ties (Kim and Meier 1991) on CFRP parallel wire tendons and for this purpose on the needed anchorage systems as described e.g. in Meier 2012.

CFRP CABLES IN REHABILITATION: EXAMPLE VERDASIO BRIDGE

Damaged State in 1997

The road bridge over Ri di Verdasio in Intragna, Switzerland was designed as a two-span reinforced concrete box girder with spans of 31.4 and 37.6 m (Figure 1). The height of the central support is 25 m. The two lane roadway is 6 m wide. The two spans of the single-cell box girder are completely pre-stressed with 6 and 8 steel tendons respectively. After only 14 years' service, a routine survey (Guidotti et al. 1999) revealed serious damage, above the "usual" wear and tear. The deck drainage pipes, which are offset by approx. 2 m on the valley side and pass through the cantilever plate at the top of the corbel, were too short and leaky. At these points, salty water ran from the roadway through the cantilever plate, along the underside of the plate, over the web and around the corner to the underside of the box girder. Deposits and traces of corrosion matching the pattern of the deck drains were
visible. A thorough investigation revealed chloride contents, relative to the mass of cement, of up to 2.8 weight % at the level of the steel reinforcement and of up to 2.0 wt.% at the level of the post-tensioning members. Chlorides penetrated up to 12 cm into the web. An exploratory window at the low point of the steel post-tensioning cables on the Locarno side, revealed the following damage: the steel reinforcement exhibited localised cross-sectional losses of up to 100%. Once the corroded cable duct had been opened, the bottom exterior steel cable showed considerable traces of pitting corrosion, which was confirmed at six further points, while the remaining three cables were still intact despite the elevated chloride content. The load-bearing safety of the bridge was verified assuming failure of both the corroded cable and a second cable and taking account of locally corroded reinforcing stirrups. Primarily due to the spare load-bearing capacity created by the complete post-tensioning, the load-bearing safety of the bridge could still be proven even in this critical state. The bridge was thoroughly overhauled in order to prevent any further progress of the corrosion which was already under way and to protect the still intact post-tensioning cables from corrosion. Although load-bearing safety could still have been guaranteed even if the corroded steel cable had failed, the bridge owner specified that the original load-bearing capacity of the bridge be re-established. The load-bearing capacity of the corroded cable on the valley side was to be replaced by external post-tensioning tendons passing through the bridge box.

Figure 1 Verdasio Bridge: Photo from Camedo side, longitudinal section and cross section

Rehabilitation with Four CFRP Cables

The required bridge retrofitting was provided with four CFRP parallel wire bundles (cables) in a polygonal arrangement (Fig. 1). Deflecting CFRP cables has in the past been considered a critical factor due to the sensitivity of the CFRP wires to shear and transverse stresses. The Ri di Verdasio Bridge was the first opportunity to use CFRP tendons as external post-tensioning cables since the successful completion of Empa's series of investigations with deflected CFRP parallel wire bundles (Maissen 2000). The parallel wire bundles were assembled by Empa and BBR Ltd. personnel in the Empa laboratories and supplied to the construction site in reel form. Four CFRP tendons, each comprising 19 CFRP wires of 5 mm in diameter, as had also been tested in the stated investigations, were used. Larger units could not have been used due to the restricted space available for the anchorages in the abutment chambers. At a degree of post-tensioning of 65%, an initial nominal tensile force of 4 x 600 kN is obtained. This corresponds to a nominal stress of 1610 MPa in the CFRP wires. The four CFRP tendons are arranged in the bridge box, on the inside of the valley side web, adjacent to each other and following a polygonal path over the entire length of the bridge (Fig. 1, longitudinal section). The external diameter of the polyethylene sheath is only 32 mm. These sheaths were not injected with mortar. The tendons are provided with tensionable anchorages on both sides. The cables are each deflected twice at approximately the third points of the span and over the pier using custom made deflectors. These deflectors comprise curved steel half-tubes and the minimum radius of deflection is 3.0 m. This radius is exactly the same as used in the full scale experiments (Maissen 2000) over the support in the centre of the girder. On site, the deflection points were concreted into the boxes through holes drilled in the roadway plate. In this way, the cables could be introduced from beneath. The deflection point over the pier initially required a hole to be drilled in the solid cross girder so that the cable and anchorage could be drawn through. The deflectors were then inserted into the hole and concreted in. Holes were also drilled in the transverse abutment beams. Transfer of the cable forces also required additional reinforcing columns to be concreted in. Lining tubes were inserted into the holes drilled in the abutment and the new columns which allowed the anchorage heads and cables subsequently to be drawn in. The CFRP tendons were then drawn into the bridge boxes through an oblique hole only 150 mm in diameter drilled in the roadway, placed in the correct position and tensioned in two stages. In medicine one would speak about a non-invasive surgery. Given the confined space in the boxes and in the abutment chambers, the low weight of the cable considerably simplified handling. The CFRP cable weighs only one fifth the amount of an equivalent steel cable.
Long-Term Monitoring

The cable force on the installed four CFRP cables is measured at each of the eight terminations with load cells based on resistance strain gauges (RSG). The load cells were calibrated in a testing machine at Empa. The scatter of this calibration showed a bandwidth of 1% of the effective forces (Anderegg et al. 2014). This accuracy is sufficient to measure relevant changes of the post-tensioning forces. The coefficient of thermal expansion for the CFRP tendons is about zero or even slightly negative. In summertime, when the temperature is high, the concrete of the bridge girder expands. Due to that the CFRP cable force is increasing. In winter it is opposite (Fig. 2). Most remarkable is that the trend lines of all cable forces are in average increasing over the observed period of time for 7.2% (Fig. 3). With steel cables one could expect a loss due to stress relaxation of steel in this range (Müller and Zetterholm 2002). The average temperature increase from 12.6 to 13.6°C (Fig. 4) might be one reason for the increase of the post-tensioning forces. However it is difficult to explain the strong increase of the force at anchorage socket 7 of cable No. 3 from 747 to 945 kN (+27%). Within the last three measurements it did no longer following the upwards trend, it is now stable at 910 kN (2440 MPa). But there is a need to check the force in the socket No. 7 soon. Interesting is that for the same cable No. 3 on the other side the force in socket No. 5 increased only from 534 to 552 kN. If the force measurement in socket No. 7 is correct, there must be such a high friction around the deviation saddles that does not allow equilibrium of the forces in cable No. 3.

![Figure 2](image2.png)  
**Figure 2** The post-tensioning force [kN] of the anchorage socket No. 3 (solid line with vertical axis left) correlates with the seasonal temperature fluctuations (dashed line with vertical axis right) versus time.

![Figure 3](image3.png)  
**Figure 3** Trend lines of the post-tensioning forces [kN] (vertical axis) of all eight anchorage sockets versus time. The average post-tensioning force for all four cables is today 674 kN (1806 MPa).

![Figure 4](image4.png)  
**Figure 4** Trend lines for the temperature [°C] measured at the meteo station in Lugano near the bridge.

![Figure 5](image5.png)  
**Figure 5** Relative displacements [mm] between load transfer media and cone of socket (vertical axis) versus time. The upper two creep curves belong to the sockets No. 1 and 4. The middle line is representing socket No. 2 and the bottom line socket No. 3.

The results of the measurements of the relative displacement between the cones of the anchorage sockets and the gradient load transfer media given in Fig. 5 are very satisfying. There is, except for the first year, no creep. The
data is representative for all eight anchor heads. This is the most important outcome of the Verdasio Bridge full scale long-term pilot project.

**CFRP CABLES IN NEW CONSTRUCTION: EXAMPLE KLEINE EMME BRIDGE**

**Geometry and Loading**

The single span girder of the 47 metre long cycle/foot bridge over the Kleine Emme River near Lucerne was post-tensioned with two CFRP tendons passed through the lower chord tube (Fig. 6). This bridge was prefabricated on site, post-tensioned and set in place as a single 130 tonne structure using a mobile caterpillar-tracked crane (Burkhardt et al. 1999). The girder performs the function of both a compressive arch and a tension tie. The solid concrete slab, up to 25 cm in thickness, constituting the compressive arch simultaneously bears longitudinal and transverse loads. The tension tie consists of a steel tube (355.6 x 30 mm) with internal CFRP post-tensioning cables. The lattice struts are of the same profile as the lower chord (ROR 139.7 x 10 to 16 mm) and arranged in a V-shape both longitudinally and transversely. While the inclination is a constant 45° in the longitudinal direction, it varies between 65° and 35° in the transverse direction as the height decreases. The connection to the concrete slab is provided in the upper truss joints by gusset plates and cross members (horizontal connection of the upper chord joints with 200 x 100 x 12 mm angle plates). To ensure stability, the upper chord is pegged to the concrete plate over its entire length. The design of the support reflects the strut geometry and, as a closed box, unites the truss structure with the support plate. Both during construction and after completion, the support box transfers the forces from the bridge bearings and eccentric post-tensioning member into the pegged concrete slab.

![Figure 6 Kleine Emme Bridge](image)

**Long-Term Monitoring**

This structure saw the first use of CFRP post-tensioning cables with fibre optic sensors integrated into the CFRP wires. Two CFRP-cables each with 91 wires of 5 mm diameter were installed inside the tube of the lower chord. Each cable was post-tensioned with 2400 kN, which corresponds to a nominal stress level of approximately 1350 MPa, corresponding to a high strain level of about 8500 μm/m on the CFRP-wires. It was of great interest to monitor the wire strains not only in the free part of the cable, but also within the anchor head. To transfer safely such high strains into the cone of the anchor head it was designed to smoothly decrease the strain in the wires from the load side to the end of the termination and to avoid stress concentration on the load side (Meier et al.2013). Fibre Bragg gratings (FBGs) inside the head allowed for the first time this design to be verified (Fig. 7). To measure the strains in the wires the FBGs were directly embedded into the CFRP-wires during the industrial pultrusion process - the first application of this kind. The directly embedded FBGs allowed locating them inside the anchor, and hence to acquire the important information about axial strain distribution during the production- and post-tensioning phase as well as during the long-term operation. In the case of this bridge, 8 of 21 FBGs failed due to high strain levels in combination with an embedding process that should be optimized. The failures occurred outside the anchor heads. Many of the resistance strain gauges (RSGs) initially attached on the wires in the free part of the cable failed already during the post-tensioning process. Therefore, additional RSGs were fitted on the steel anchor heads—to obtain redundant information about force variation (Fig. 7). Similar to the situation at the Verdasio bridge there is also in the case of Kleine Emme Bridge a strong correlation between temperature cycles and the cycles of the post-tensioning force. Opposite to Verdasio Bridge there is at Kleine Emme Bridge no increase of the average forces. The trend line for the continuously measured forces is horizontal (Fig. 8). Similar like in Fig. 5 there is also for the anchorage heads of this 91-wire parallel wire bundles beside the first two years no creep. This is the most important outcome of the Kleine Emme Bridge full scale long-term pilot project.
Figure 7 Longitudinal section of the anchor head with RSG and FBG sensors on the steel sleeves and the CFRP wires inside and outside of the anchor head.

Figure 8 The post-tensioning force [kN] of a 91-wire cable of Kleine Emme Bridge (red line with vertical axis left) correlates similar like in Fig. 2 with the seasonal temperature fluctuations (blue line with vertical axis right) versus time. Opposite to Fig. 3 the trend line (dashed horizontal red line) shows no increase of the post-tensioning force. The average post-tensioning force of 2410 kN is well matching the nominal force of 2400 kN (1350 MPa). Similar to Fig. 4 there is also an increase in temperature (dashed blue line).

CONCLUSIONS

On the described two bridges, post-tensioning forces and displacements were measured with different measuring techniques. Well-established RSGs and LVDTs as well as newly developed FBG sensors have been in service for 18 years. To obtain information regarding the long-term reliability, reference sensors and redundant sensing systems have been installed, allowing one to differentiate between sensor drift and changes in the performance of the CFRP-cables. Additional long-term laboratory experiments have helped to validate sensing systems. Well planned sensing systems, verified methods and careful installation resulted in reliable long-term monitoring systems needed to proof the long-term reliability of CFRP cables. This allowed to demonstrate the very high reliability of the CFRP tendons under high stresses over a period of 18 years. The key element for the development of such tendons was from the beginning the anchorage system. With the development of a gradient load transfer media (LTM) based on aluminium oxide pellets and epoxy resin it is possible to make use of the high strength of the CFR wires and to avoid creep failure of the socket (Kim and Meier 1991).

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PRESTRESSING SYSTEM OF MULTI-PLY FRP SHEETS FOR STRUCTURAL STRENGTHENING

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2 Vanke Co., Ltd, China.

ABSTRACT

External prestressing is an appropriate technique for improving the behavior of existing structural members under service loads and increasing the ultimate strength. In view of the insufficiency of prestress provided by single layer of FRP sheet, a novel prestressing system was developed for multi-ply sheets. It consists of an anchor device and a tension one apart from FRP sheet which is combined with the devices by interlocking. Two experiments were conducted to explore the ultimate capacity for the prestressing system of 4-ply and 100-mm-wide carbon fibre-reinforced polymer (CFRP) sheets. The results showed that this prestressing system can effectively anchor multi-ply sheets of CFRP resulting in rupture and achieve the fibre strength utilization up to 83.97%. Then three steel I-beams were prestressed by the same system with different lengths and the response of prestressing are measured. It was found that the test results accorded well with the theoretical values for deflection and strain of the I-beams and uniformity of the strap strains was improved with increase of the strap length. At last further application was simply introduced for strengthening of reinforced concrete box girders with the prestressing system of more plies CFRP sheets.

KEYWORDS

Prestressing system, FRP, structural strengthening, engineering application.

INTRODUCTION

Due to high strength-to-weight ratio, good corrosion resistance and ease of installation, externally bonded fibre-reinforced polymer (FRP) straps have been widely used for the flexural strengthening of RC and steel structures. Extensive research has shown that the controlling failure mode of such a strengthened structural member often involves the premature debonding of the FRP end from substrate in a brittle manner (Smith and Teng 2001, 2002a, 2002b). Furthermore, stress hysteresis of the FRP strap generally exists in the traditional flexural strengthening, which results that the FRP cannot play its high strength characteristics unless deformation of the strengthened beam is very large. To prevent the premature debonding, researchers have actively explored the end anchoring techniques of FRP, such as anchor spikes (Lam and Teng 2001), metallic plates (Wu and Huang 2008), WSGG anchor (Zhuo et al. 2009) and double-plate anchor (Belarbi et al. 2012). In order to reduce the stress hysteresis of the FRP strap, limited literatures have reported prestressing techniques (TC Triantafillou and N Deskovic 1992; RG Wight et al. 2001; Colombi et al. 2003; Raafat El-Hacha et al. 2003; YJ Kim et al. 2008; Huawen et al. 2010; Yang and Li 2010; Shang et al. 2003; Elyas Ghafoori et al. 2015) which meanwhile reduce deflection of the member and control cracks. However, existing investigation of prestressing with end anchorage focused on FRP plates, and few on FRP sheet, particularly on multi-ply sheets. In fact, a single layer of FRP sheet can just supply low pretension for the strengthened element, leading to insignificant effect of strengthening. It is so necessary to develop a prestressing system for multi-ply sheets. This paper introduces such a new prestressing system and its application for flexural strengthening of steel or RC structures.

PRESTRESSING SYSTEM AND EXPERIMENTS

System constitution

The prestressing system consists of an anchor device, a tension device, and multi-ply FRP sheets, as shown in Figure 1. The anchor device, used to anchor one end of the FRP strap, is assembled by two anchor plates and four anchored bolts. The anchor plate invented by the first author (Zhou et al. 2012) can make the FRP strap totally fixed just by wrapping round the plate in a special method. The tension device, including an anchor box, a position
plate, four anchored bolts, and two pull rods, aims to anchor the other end of the strap and exert prestress. Either manual screwing or mechanical jacking was tried to apply the prestrain and proved feasible. During pretension the prestressing value is controlled by strain gauges on the pull-rod surface or reading meter of the jack, as shown in Figures 2a-b.

Figure 1 Prestressing system for multi-ply FRP sheets

Material tests

Toray carbon fibre-reinforced polymer (CFRP) sheets and Araldite epoxy resins were adopted in experiments. According to Chinese code for carbon fibre sheet for strengthening and restoring structures (GB/T 3354-1999), eight straps of CFRP 12.5mm wide were formed by embedding continuous fibres in epoxy resin which bonded the fibres together. Then, these specimens were tested by tension testing machine after 7 days curing, and testing result is shown in Table 1.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Thickness (mm)</th>
<th>Tensile strength (MPa)</th>
<th>Tensile modulus (MPa)</th>
<th>Compressive strength (MPa)</th>
<th>Ultimate strain (με)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP</td>
<td>0.167</td>
<td>3606</td>
<td>2.5×10^5</td>
<td>–</td>
<td>14424</td>
</tr>
<tr>
<td>Epoxy resins</td>
<td>–</td>
<td>55</td>
<td>3.5×10^3</td>
<td>88</td>
<td>–</td>
</tr>
</tbody>
</table>

Tests of prestressing system

A prestressing system for 4-ply and 100-mm-wide CFRP was designed and manufactured. Two prestressed CFRP specimens numbered P1 and P2 were made for the experiment and they had the same thickness of 0.667 mm, length of 450 mm, and width of 98 mm, as shown in Figure 3. Then the ultimate capacity experiments of the specimens were carried out to observe the failure mode of the CFRP strap and verify the reliability of the anchor device and the tension device, as shown in Figure 4. In loading process, the force of 10 kN was firstly preloaded with a rate of 5 kN/min, then 180 kN was exerted with the same rate and sustained for 10 minutes. After 180 kN, further tension was applied with a rate of 3 kN/min until the specimen failed.
Results and discussions

Figure 5 shows the damage diagrams of P1 and P2. It can be seen that their failure modes are both the ruptures of the CFRP strap. The fracture location of P1 is close to the end of the anchor plate, and that of P2 is in the central part of the CFRP strap. However, the CFRP fibers in the anchor plate and the anchor box remain intact. As shown in Table 2, in average the ultimate tension of the CFRP strap is 200.3 KN, correspondingly the system strength is 3028 MPa and the utilization ratio of material strength is 83.97%. Furthermore, the average sliding distances of CFRP at the anchor-plate end and the anchor-box end are zero and 1.7 mm respectively. All these results indicate that this prestressing system is feasible and capable to anchor CFRP straps and achieve rupture failure.

![Figure 3 Specimen of CFRP sheets with end anchorage](image3)
![Figure 4 Ultimate tension testing](image4)

(a) P1

(b) P2

Figure 5 Damage diagrams

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Sliding distance of CFRP at the anchor-plate end</th>
<th>Sliding distance of CFRP at the anchor-box end</th>
<th>Ultimate tension (kN)</th>
<th>System strength (MPa)</th>
<th>Material strength (MPa)</th>
<th>Strength utilization ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>0 mm</td>
<td>1.6 mm</td>
<td>193.7</td>
<td>2959</td>
<td>3606</td>
<td>82.05</td>
</tr>
<tr>
<td>P2</td>
<td>0 mm</td>
<td>1.8 mm</td>
<td>206.9</td>
<td>3097</td>
<td>3606</td>
<td>85.89</td>
</tr>
<tr>
<td>Average</td>
<td>0 mm</td>
<td>1.7 mm</td>
<td>200.3</td>
<td>3028</td>
<td>3606</td>
<td>83.97</td>
</tr>
</tbody>
</table>

Note: Strength utilization ratio=Equivalent strength / Tensile strength×100%

PRESTRESSING RESPONSE OF STEEL BEAMS

Three steel I-beams with a length of 3250mm and a height of 250mm were strengthened by prestressed CFRP straps based on this system to verify the applicability of the prestressing method and explore the influence of the strap length on the uniformity of CFRP strain distribution across the width. The strap lengths were 1000mm, 1500mm and 2000mm respectively and the strengthened I-beams were accordingly numbered P10, P15 and P20. All the straps had a width of 98mm and a thickness of 0.667mm. In each experiment pretension of 110kN was applied by the manual screwing method, as shown in Figure 2b. Dial indicators were arranged on the midspan and the two supports of the I-beams to monitor the change of deflection. Meanwhile, strain gauges were attached to the middle of the upper flange of I-beams, and three gauges were arranged on each section of CFRP strap respectively located at section A close to the anchor plate, B in the middle of the CFRP, and C close to the anchor box, as shown in Figure 6.
The theoretical values of deflection and strain of the strengthened beam of steel are calculated based on equation (1) and equation (2) respectively.

\[ \Delta = \frac{1}{EI} \times \frac{1}{2} \times \frac{L}{4} \times T \times e = \frac{T_e L}{8EI} \]  
\[ \varepsilon = \frac{\sigma}{E} = \frac{I + A}{E} \]

where \( \Delta \) and \( \varepsilon \) are the deflection and strain of the I-beam; \( E, I, L \) and \( A \) are the elastic modulus, moment of inertia, length, and sectional area of the I-beam, respectively; \( T, e, M \) and \( y \) denote the pretension, its eccentricity, bending moment, and distance between calculated position and neutral axis.

Comparisons between the theoretical and experimental values of deflection and strain are shown in Figure 7 and Figure 8 respectively. It can be seen that the measured values of strain are very coincident with the predicted results, and the relative errors between the calculated and experimental values of deflection and strain are both about 5%. These results demonstrate the prestressing system is applicable to steel I-beams. Furthermore, coefficients of variation are adopted to express the influence of the strap length on the uniformity of CFRP strain distribution over its width. Low coefficient implies relatively even distribution of strains at a section. It can be seen from Table 3 that the strain distribution at any section will become uniform with the increase of the strap length. In addition, the variation of strain at section B, i.e. in the middle, is less than that at section A and C near the anchorage ends.

![Figure 6 Prestressing system of FRP strap on a steel beam](image_url)

![Figure 7 Comparisons between the calculated and experimental values of deflection](image_url)

![Figure 8 Comparisons between the calculated and experimental values of strain](image_url)
Table 3 Coefficient of variation for CFRP strain at different sections

<table>
<thead>
<tr>
<th>Load (kN)</th>
<th>Position A (%)</th>
<th>Position B (%)</th>
<th>Position C (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>P10</td>
<td>P15</td>
<td>P20</td>
</tr>
<tr>
<td>24</td>
<td>44.7</td>
<td>24.4</td>
<td>22.1</td>
</tr>
<tr>
<td>45</td>
<td>43.6</td>
<td>28.5</td>
<td>25.8</td>
</tr>
<tr>
<td>65</td>
<td>44.4</td>
<td>28.9</td>
<td>22.5</td>
</tr>
</tbody>
</table>

Note: Coefficient of variation = the standard deviation / the mean x 100%

ENGINEERING APPLICATION IN CONCRETE GIRDER

The prestressing system above-mentioned was also applied for strengthening cracked RC box girder of some bridges in Shanxi and Shaanxi, two provinces in China. The CFRP straps were 6 or 8 layers and had a length of about 25000mm and a width of 100mm. The exerted pretension value of each strap was 100-150kN. The construction procedure of this system mainly contained four parts: cutting the CFRP, assembling components, applying prestress, and anticorrosion processing, as shown in Figure 9. After the bridge was strengthened, the inspection results demonstrated that the deformation of the strengthened box girders and the development of their flexural cracks were obviously controlled, which proved the applicability of the proposed system and further validated the feasibility of the construction process.

(a) Cutting the CFRP  (b) Assembling components  (c) Applying prestress load  (d) Anticorrosion processing

Figure 9 Construction of flexural strengthening of RC girder with prestressed CFRP sheets

CONCLUSIONS

This paper introduces an innovative prestressing system with end anchorage for multi-ply FRP sheets to prevent the premature debonding and in particular reduce the stress hysteresis of FRP in structural strengthening. The multi-ply sheets are combined at two ends with patented interlocking devices, which provide brand-new ways for anchoring FRP sheets. The experimental results indicate that this prestressing system is capable to anchor multi-ply CFRP sheets and achieve rupture failure, which supplies fiber strength utilization to 83.97%. The uniformity of strap strains for the longer FRP strap is better than that for the shorter one and strains at the middle sections of different-length straps are more uniform than strains at other sections. Their application to both steel I-beams and concrete Girders shows that the construction procedure for the system is feasible in practical engineering.

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REFERENCES


EXPERIMENTAL INVESTIGATION ON RC BEAMS WITH PRESTRESSED NSM SYSTEM

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ABSTRACT

Recently, rehabilitation of concrete structures with fiber reinforced polymer (FRP) has grown to be an extensively used system in most parts of the world. The main aim for this is that it is possible to obtain a suitable strengthening effect with a relatively small work effort. Nevertheless, when strengthening a structure with FRP, it is often not possible to make full use of the FRP due to de-bonding ruptures. This disadvantage of early de-bonding can be overcome by using a near surface mounted (NSM) strengthening system. In addition, prestressing can be more effectively introduced in the NSM strengthening system than in another strengthening method. The effectiveness of strengthening reinforced concrete (RC) beams with prestressed NSM carbon fiber reinforced polymer (CFRP) systems was investigated. Three RC beams (600 mm deep by 400 mm wide by 6400 mm long) were tested under four-point loading test. One beam was kept un-strengthened as a control beam. Another was strengthened with a pre-tension NSM CFRP bar. The other was strengthened with post-tension NSM CFRP bar. The test results showed that strengthening with prestressed NSM beam enhanced the flexural behavior of the beam compared to that of the control beam. The test results indicated that the prestressed NSM beams more effectively improved the cracking, yielding and ultimate loads than the control beam. The post-tension NSM systems was determined that enhances the beam performance under concrete cracking, steel yielding and ultimate loads.

KEYWORDS

Prestressed near surface mounted, carbon fiber reinforced polymer, RC beam, strengthening, four-point loading test

INTRODUCTION

Situations that cannot be predicted during planning often occur in the lifetimes of concrete structures. The durability of a concrete structure such as a bridge is decreased by construction defects, vehicle loads, environmental changes, and material characteristics. Bridges located on seashores are exposed to severe environments, which can lead to the corrosion of prestressed steel bars within the concrete. Corroded prestressed steel bars can cause structural problems because of the decreased premressing forces they provide. Recently, various studies have been performed on the maintenance of concrete structures. Fiber-reinforced polymer (FRP) bars, including glass and carbon fibers with advantageous corrosion resistances, have been researched extensively. Although FRP has the weakness of brittle failure, it is superior in strength, weight, durability, and resistance to creep and fatigue compared to existing construction materials. The lightweight material is also easy to use in the field. However, concrete structures externally bonded with FRP have low fire resistance and high vulnerability to vehicle collisions and bond failure. In order to compensate for the instability of externally bonded systems, near-surface mounted (NSM) systems, which embed FRP into concrete structures, have been investigated. NSM systems have been studied experimentally and analytically based on the types of FRP bars and characteristics of the materials used. Standard methods for bonding FRP bars to concrete have been proposed (ACI 2001; CSA 2002; JSCE 2007).

De Lorenzis et al. tested NSM systems using parameters of the shape of the FRP, type of filler, bond length, and size of the groove, and found that the bonding strength of the test specimen using mortar as filler was lower than that of the test specimen using epoxy (De Lorenzis et al. 2002a, 2002b). The results also showed that, when the sand was added to the filler, the viscosity could be controlled by the volumetric expansion of the epoxy. The epoxy lowered the coefficient of expansion to control the glass transition temperature (De Lorenzis et al. 2002b). Benmokrane et al. (2002) performed experimental research on the bond characteristics and fracture shapes of FRP
bars by embedding FRP bars in aluminum and PVC sleeves. Shraky et al. (2013a, 2013b) studied the bonding characteristics between filler and FRP bars by executing pullout tests with the groove size, filler, and type of FRP as parameters. Hassan et al. (2002) fabricated a half-scale reinforced concrete (RC) girder and executed load tests by placing carbon FRP (CFRP) bars and plates on the surface of the girder and evaluating the bonding characteristics of the CFRP with increases in loading. Casadei et al. (2006) produced an 11.0-m prestressed concrete (PSC) I girder, strengthened with both a prestressed external bond system and an NSM system, and performed four-point loading tests. In the study, the strengthening effects and failure modes of the NSM-strengthened girder were analyzed for varying prestressing forces and types of FRP. Al-Mahmoud et al. (2010) fabricated a 2.8-m RC beam with an NSM system and performed a four-point loading and cantilever loading tests. The test results, cracking failure, and bonding failure were compared and verified with a finite-element (FE) model. The NSM system places CFRP bars in the concrete to introduce prestressing, thereby improving the performance of the concrete structure. As stated in the literature reviews above, studies on NSM systems have focused on bonding and bending stresses. The flexural behavior of RC beams with prestressed NSM systems can ameliorate serviceability concerns, such as the deflections at different load steps in RC beams (Badawi, M., and Soudki, K 2009). Prestressed NSM systems can prevent premature de-bonding at the surfaces of concrete specimens. In addition, prestressed NSM systems significantly improve the crack, yield, and ultimate loads while reducing crack widths in strengthened beams, compared to those observed in non-prestressed strengthened RC beams. A similar study demonstrated that load cycling at room temperature (~20°C) could weaken the CFRP-concrete bond. Several analytical processes for studying and FE modeling of RC beams strengthened with prestressed NSM systems have been proposed (Rasheed et al. 2006; Soliman et al 2010).

The number of experimental studies on the strengthening of RC beams with prestressed NSM systems is limited. Most studies on prestressed NSM systems have adopted the pre-tensioning method. In this study, the flexural behavior of RC beams strengthened with both pre-and post-tensioning NSM systems is analyzed.

EXPERIMENTAL PROGRAM

Material and methods

In this study, a 6.4-m-long RC beam was fabricated for testing by four-point loading. The height, width, and concrete strength of the RC beam were 600 mm, 400 mm, and 40 MPa, respectively. The surfaces of each CFRP bar were processed by sand coating. For sand coating, sand was sprayed after applying a coating agent onto the surface of the ground CFRP bars. Figure 1 shows the prestressed NSM system dimensions.

The experimental parameters are the prestressing method, number of bars, as shown in Table 1. Five RC beams were fabricated. The first segment of the specimen name represents the prestressing method: “Pre” stands for pre-tensioning and “Post” stands for post-tensioning. The second segment represents the prestressing force. In the FRP strengthening method, prestressing forces of 40-60% of the tensile strength of the FRP bars have been proposed in the ACI committee. In this study, a prestressing force of 120 kN was applied to the CFRP bar, ~50% of the tensile strength of the CFRP bars. The third segment indicates the reinforcement length, which was selected as 83% (5.0 m) of the length of the RC beam (6.4 m). The fourth segment represents the type of filler: “M” stands for mortar. The fifth segment represents the surface treatment of the CFRP bar. “SC” indicates sand coating. The sixth segment indicates the number of FRP bars: “1” means one-line strengthening and “2” means two-line strengthening. The diameter of each CFRP bar was 10 mm, and the tensile strength and elastic modulus of CFRP bar was 3290 MPa, 171 GPa, respectively.

![Figure 1 RC test beam details](image-url)
Table 1 Experimental parameters

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Prestressing Method</th>
<th>Reinforcement Length (m)</th>
<th>Filler Material</th>
<th>Number of Groove</th>
<th>Surface Treatment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Pre-50-83-M-SC-1</td>
<td>Pre-Tension</td>
<td>5</td>
<td>Mortar</td>
<td>1</td>
<td>Sand coating</td>
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<tr>
<td>Pre-50-83-M-SC-2</td>
<td>Pre-Tension</td>
<td>5</td>
<td>Mortar</td>
<td>2</td>
<td>Sand coating</td>
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<td>Post-Tension</td>
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<td>Mortar</td>
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<tr>
<td>Post-50-83-M-SC-2</td>
<td>Post-Tension</td>
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<td>Mortar</td>
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<td>Sand coating</td>
</tr>
</tbody>
</table>

In this study, rebars are assembled as shown in Figure 2(a). The assembled rebar was inserted in a produced mold and arranged with precise spacing (see Figure 2(b)). Figure 2(c) depicts the pouring of concrete with a compressive strength of at least 40 MPa. After pouring, vibration compaction was performed up to three times depending on the height. After 14 days of curing, a groove measuring 30 mm in width and 40 mm in depth was fabricated in the concrete, as shown in Figure 2(d). Figure 2(e) shows the process of introducing prestressing using the pre-tensioning method. Figure 2(f) shows the process of anchoring and installing CFRP bars using the post-tensioning method. The process of applying the post-tensioning method of prestressing to the CFRP bars is shown in Figure 2(g). The fabricated groove was filled with epoxy and mortar, which were then cured (see Figure 2(h)).

(a) Assemble rebar  
(b) Assemble the mold  
(c) Cure the concrete  
(d) Form the groove

(e) Apply pre-tensioning the CFRP bar  
(f) Place anchorage device  
(g) Apply post-tensioning the CFRP bar  
(h) Inject the filler

Figure 2 Fabrication process of specimen

Figure 3 shows the layout of the test setup. In this experiment, the load was applied stepwise using a universal testing machine with a capacity of 200 kN. The loading positions were located 500 mm away from the mid-span on both sides of the RC beams. The displacement of the RC beam under loading was measured by a displacement transducer, which was installed at the mid-span. The displacement control method was used for the loading. A rate of 0.03 mm/s was used for the first 30 mm; after reaching this length, the rate was increased to 0.1 mm/s.
EXPERIMENTAL RESULTS

In this study, 14 days after the filler in the groove was cured, the prestressing force was removed from the RC beam strengthened with the pre-tensioned NSM system. The prestressing loss was measured at the center of the CFRP bar, the CFRP bar slip was measured at both ends of the CFRP bar. After the experiment, the CFRP bars were checked for possible slip occurrence. Slip of about 20 mm occurred in the CFRP bars, for all RC beams strengthened with pre-tensioning NSM systems after the prestressing forces were removed. The prestressing forces and prestress force losses of the CFRP bars were measured with strain gauges installed in the centers of the CFRP bars. The prestress losses of the pre-tensioned NSM-strengthened RC beams are summarized in Table 2. The losses of the Pre-50-83-M-SC-1 and Pre-50-83-M-SC-2 are 27.1 and 39.4%, respectively. As the number of bars in the pre-tensioned NSM-strengthened RC beam is increased, the prestress force losses of the bars are increased. From the results, increases in the number of the CFRP bars in an RC beam are observed to cause mutual interference between the CFRP bars and increase the prestress force losses.

Table 3 shows the comparisons of the load level for strengthened RC beams. The load levels of the RC beam are classified as the concrete cracking load, steel yielding load, and maximum load of the RC beam. For pre-tension NSM strengthened RC beams, the Pre-50-83-M-SC-2 specimen shows concrete cracking, steel yielding, and maximum loads of 17.7%, 8.4%, and 2.8% higher than those of the Pre-50-83-M-SC-1 specimen. The strengthening effect on the RC beam in the pre-tensioned NSM-strengthened RC beam is improved as the number of CFRP bars is increased. However, the prestress force loss is increased as the number of CFRP bars in the pre-tensioned NSM-strengthened RC beam without anchorage is increased; therefore, the strengthening effect of RC beam is not considerably improved.

The experimental results of post-tension NSM strengthened RC beam according to the number of CFRP bars reveal that the Post-50-83-M-SC-1 specimen shows concrete cracking, steel yielding, and maximum loads that are 17.9%, 29.4%, and 22.3% higher than those of the Post-50-83-M-SC-2 specimen, respectively. The strengthening effect on the RC beam for all load levels is increased by approximately 20% with the post-tensioned NSM strengthening systems using two CFRP bars, relative to those with one CFRP bar. In the post-tensioned NSM-strengthened RC beams, CFRP bar slip does not occur because anchorage systems are placed in the RC beams. Therefore, the prestress force loss of the CFRP bar in the post-tensioned NSM-strengthened RC beam is less than that for the pre-tensioned NSM-strengthened RC beam. This trend suggests that the post-tensioned NSM strengthening system minimizes the prestress force loss of the CFRP, thereby improving the strengthening effect for the RC beam at all load levels.

Table 2: Prestress loss of pre-tension NSM strengthening RC beams

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Strain (με)</th>
<th>Effect strain (με)</th>
<th>Effect prestressing force (kN)</th>
<th>Prestress force loss (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-50-83-M-SC-1</td>
<td>8556</td>
<td>6725</td>
<td>88.0</td>
<td>27.1</td>
</tr>
<tr>
<td>Pre-50-83-M-SC-2</td>
<td>8181</td>
<td>5592</td>
<td>72.9</td>
<td>39.4</td>
</tr>
</tbody>
</table>
Figure 4 shows the load–displacement relationships according to the pre-stressing methods. The concrete cracking load, steel yielding load, and maximum load of the RC beam in the Post-50-83-M-SC-1 specimen are greater by 50.0%, 18.9%, and 30.8%, respectively, than those in the Pre-50-83-M-SC-1 specimen. The Post-50-83-M-SC-2 specimen shows increases of the concrete cracking load by 50.3%, steel yielding load by 42.0%, and maximum load of the RC beam by 55.5%, relative to the Pre-50-83-M-SC-2 specimen. The strengthening effect on the RC beam is not considerably improved by the Pre-50-83-M-SC-2 system because the prestress loss of the pre-tensioned RC beam without anchorage is high. However, the post-tensioned NSM-strengthened RC beam minimizes the prestress loss by anchorage. Accordingly, the strengthening effect on the RC beam by the post-tensioned NSM strengthening system is significantly improved when the number of CFRP bars is increased.

Table 3 Experimental results: comparisons of the load level

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Concrete crack load (kN)</th>
<th>Steel yielding load (kN)</th>
<th>Maximum load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>68.3</td>
<td>152.5</td>
<td>193.2</td>
</tr>
<tr>
<td>Pre-50-83-M-SC-1</td>
<td>75.9</td>
<td>185.3</td>
<td>228.6</td>
</tr>
<tr>
<td>Pre-50-83-M-SC-2</td>
<td>89.4</td>
<td>200.9</td>
<td>235.1</td>
</tr>
<tr>
<td>Post-50-83-M-SC-1</td>
<td>113.9</td>
<td>220.4</td>
<td>298.9</td>
</tr>
<tr>
<td>Post-50-83-M-SC-2</td>
<td>134.4</td>
<td>285.2</td>
<td>365.5</td>
</tr>
</tbody>
</table>

Figure 4 Load–comparison between pre-tensioned and post-tensioned beams.
CONCLUSIONS

In this study, in order to analyze the prestressed NSM system, RC beams of 6.4 m in length were fabricated and tested by four-point loading. The following conclusions were obtained:

The test results showed that the prestressed NSM beams more effectively improved the cracking, yielding and ultimate loads than the control beam. The strengthening effect of the pre-tensioned NSM strengthening system on the RC beam in which anchorage was not used was insignificant because of the considerable prestress loss. The strengthening effect on the RC beam was improved by the post-tensioned NSM strengthening for which anchorage was installed, because the anchorage minimized the prestress loss of the CFRP bars. Accordingly, the installation of anchorage is necessary for the prestressed NSM system in order to prevent the slippage of the FRP bars and to minimize the prestress loss. The strengthening effect of the post-tensioned NSM strengthening on the RC beam was improved as the number of CFRP bars was increased. However, the strengthening effect of the pre-tension NSM strengthening on the RC beam without anchorage was not improved, even for increased numbers of CFRP bars. CFRP bar slip did not occur for any number of CFRP bars in the post-tensioned NSM-strengthened RC beam because of the anchorage constraints.

ACKNOWLEDGMENTS

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PRESTRESSED NSM CFRP FOR STRENGTHENING A CONSTRUCTED BRIDGE: ANCHORAGE BEHAVIOR FROM MACRO TO MICROSCALE PERSPECTIVES

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ABSTRACT

This paper presents a modeling approach for prestressed concrete bridge girders strengthened with post-tensioned near-surface-mounted (NSM) carbon fiber reinforced polymer (CFRP) strips. Three methods are employed to predict the macro-, meso- and microscale behavior of anchorage zones when the CFRP strips are post-tensioned, including finite element, discrete-object physics, and analytical models. The multiscale integration used is based on domain decomposition in connection with information-passing. The macroscale finite element model provides strain softening in the vicinity of the anchorage, which is related to smaller scale responses depending upon the post-tensioning level of the CFRP.

KEYWORDS
Carbon fiber reinforced polymer (CFRP), modeling, multiscale, near-surface mounted (NSM), prestress

INTRODUCTION

Strengthening with carbon fiber reinforced polymer (CFRP) composites is proven technology to upgrade the performance of existing bridge members. Many benefits are involved, such as reasonable construction costs, reduced long-term maintenance expenses, and minimal traffic disruption (Lopez and Nanni 2006). Conventionally, two types of CFRP-strengthening methods are implemented, namely, externally bonded (EB) CFRP and near-surface-mounted (NSM) CFRP. The EB method is broadly used in the rehabilitation community, whereas the NSM method is relatively less used because of its short application history. Unlike traditional CFRP-strengthening, prestressed CFRP may be employed when upgrading existing structural members (Wight et al. 2001). The potential of prestressed NSM CFRP has been recently studied by several researchers. Badawi and Soudki (2009) tested reinforced concrete beams strengthened with post-tensioned NSM CFRP rods. The jacking system utilized was composed of clamping anchors, a jack, and load-bearing brackets. The CFRP was tensioned up to 60% of its tensile strength. Although minor slip was observed during post-tensioning, strain compatibility was well preserved across the beam section. As the strengthened beams were loaded in flexure, concrete crushing or CFRP rupture took place. Oudah and El-Hacha (2012) conducted a fatigue test to assess the performance of concrete beams strengthened with post-tensioned NSM CFRP strips. Fatigue cycles were applied at 2-4 Hz along with a stress range of 70% of the yield stress of the reinforcing steel. The variation of deflection and stiffness was monitored to evaluate fatigue-induced damage. Ye et al. (2014) loaded one-way slabs retrofitted with post-tensioned NSM CFRP bars until failure. The level of post-tensioning was 30% of the CFRP’s strength. At transfer, a 10% prestress loss was noticed. On account of the NSM CFRP, the strengthened slabs’ cracking load was improved by 60%; however, their ductility at failure was reduced.

The state-of-the-art of post-tensioned NSM CFRP includes experimental investigations in tandem with macroscale modeling. There is a need for examining various scale responses of NSM CFRP strengthening systems, and this effort is not available in most cases. It is an important subject because the damage of a strengthened member initiates at microscale and propagates to larger scales, which cannot be obtained from conventional macroscale modeling methods. As such, a comprehensive understanding of a post-tensioned NSM CFRP system can be achieved by examining the multiscale behavior of the system. As far as multiscale modeling is concerned, sequential coupling or information-passing is typical (Weinan 2011), which is formulated with a representative volume element (RVE). The information obtained from a smaller scale response is passed to a larger scale to
connect their distinct scale behavior, or from larger scale to smaller scale. This paper discusses a multiscale modeling approach to understand the response of anchorage concrete when NSM CFRP strips are post-tensioned.

**RESEARCH SIGNIFICANCE**

The anchorage concrete of a bridge girder may experience variable scales of damage, when subjected to a post-tensioning force applied to NSM CFRP strips. From a microscale standpoint, damage nucleation and progression lead to meso- and macroscale problems. These damage propagation mechanisms can influence the functionality of a post-tensioned NSM CFRP system installed to a bridge girder. Technical investigations are necessary to comprehend the significance of multiscale damage, although the macroscale behavior of concrete members with NSM CFRP is well elucidated. The proposed modeling approach connects traditional structure-level knowledge with insights from smaller scales.

**DESCRIPTION OF A POST-TENSIONING SYSTEM FOR NSM CFRP**

Three benchmark prestressed concrete girder bridges ($L = 25$ m, $30$ m, and $35$ m) were considered in this research program, as shown in Fig. 1(a). The girders had a compressive concrete strength of $40$ MPa and standard prestressing steel strands ($f_{pu} = 1,860$ MPa and $E_p = 190$ GPa where $f_{pu}$ and $E_p$ are the ultimate strength and elastic modulus of the steel, respectively) that were tensioned up to $68\% f_{pu}$. CFRP strips ($A_f = 700$ mm$^2$ where $A_f$ is the cross-sectional area of the CFRP) included a nominal tensile strength of $2,500$ MPa and a modulus of $165$ GPa, and were post-tensioned up to $60\%$ of the strength. Figure 1(b) exhibits the sequence of a proposed strengthening technique (this method was patented: 10-0653632, 10-1005347, and 10-1083626): installing an anchor plate with high-strength bolts, tensioning NSM CFRP embedded in a precut groove using a jacking apparatus, transferring the post-tensioning force to the beam, and grouting the groove and anchorage to improve aesthetics.

**MULTISCALE MODELING APPROACH**

Figure 2 illustrates the concept of a multiscale modeling approach for the proposed strengthening system. When the anchorage is tensioned by the NSM CFRP, concrete near the anchor bolts is under complex stress states in partial compression and tension. Of interest is the tensile behavior of the concrete because premature cracking occurs. The tension zone of the concrete encompasses multiple components (i.e., aggregates surrounded by cement paste), and each inclusion has a weak link called the interfacial transition zone (ITZ) where microscale damage occurs.

![Figure 1](image1.png)
(a) Benchmark bridge girders
(b) Strengthening sequence

![Figure 2](image2.png)
Figure 2 Concept of multiscale modeling
Figure 3 Multiscale modeling framework: (a) macroscale finite element model; (b) mesoscale discrete-object physics model; (c) microscale analytical model

**Macroscale model**

Finite element modeling was conducted with a commercial software package. The concrete, and prestressing strands and CFRP strips were modeled by elastic shells and spars [Fig. 3(a)], respectively, with the foregoing material properties. By constraining nodes near both ends of the girders, simple supports were represented. The prestressing forces of the strands and strips were applied based on initial strains. Further details about model development and validation are available in Kim et al. (2014).

**Mesoscale model**

The behavior of the anchorage concrete was modeled by a discrete-object physics approach, as shown in Fig. 3(b). This model was constructed according to Newton’s second law in conjunction with the interaction between element particles [Fig. 4(a)]. The 125-mm cube model had an element size of 2 mm and a Φ25 mm steel bolt. The element interaction model [Fig. 4(b)] consisted of the following properties: normal and transverse directional virtual springs ($k_n$ and $k_t$, respectively), normal and transverse directional damping ratios ($c_n$ and $c_t$, respectively), normal and transverse directional bonding ($b_n$ and $b_t$, respectively), and static and kinetic friction coefficients ($f_s$ and $f_k$, respectively). The constructed model was solved by parallel computing technologies (Gitman et al. 2008) with the aid of a supercomputer.

**Microscale model**

The microscale behavior of a single inclusion or element [Fig. 3(c)] was examined by an analytical model. The constitutive relationship of the inclusion may be expressed as:

$$\sigma_{ij} = (1 - D_{\text{micro}}) C_{ijkl} \varepsilon_{kl}$$

(1)
where $\sigma_{ij}$ is the stress vector of the ITZ with corresponding strain $\varepsilon_{ij}$; $D_{\text{micro}}$ is the microscale damage index; and $C_{ijkl}$ is the stiffness tensor. The damage index may be obtained by (Marzars 1981):

$$D_{\text{micro}} = 1 - \left( \frac{\varepsilon_{cr}}{\varepsilon_{po}} \right) \left( 1 - A \exp \left[ B \left( \varepsilon_{m} - \varepsilon_{po} \right) \right] \right) \text{ for } \varepsilon > \varepsilon_{cr} \tag{2}$$

where $\varepsilon_{cr}$ and $\varepsilon_{po}$ are the strain corresponding to the peak stress and the post-peak strain, respectively; $A$ and $B$ are empirical constants for the residual stress and post-peak softening slope, respectively; and $\varepsilon_{m}$ is the strain at an arbitrary post-peak stress. After some manipulation of a stress-strain relationship for concrete (Hsu and Zhang 1996), the empirical constants can be determined:

$$A = \frac{\sigma_{po}}{f_r} \text{ and } B = \frac{(f_r - \sigma_{po})}{(\varepsilon_{m} - \varepsilon_{cr})} \tag{3}$$

where $\sigma_{po}$ is the residual post-peak stress and $f_r$ is the modulus of rupture. Finally, the stress states of the ITZ in radial ($\sigma_r$) and circumferential ($\sigma_\theta$) directions are calculated (Ugural and Fenster 1995):

$$\sigma_r = \sigma_{xx} \cos^2 \theta + \sigma_{yy} \sin^2 \theta + 2\sigma_{xy} \sin \theta \cos \theta \tag{4}$$

$$\sigma_\theta = \sigma_{xx} \sin^2 \theta + \sigma_{yy} \cos^2 \theta - 2\sigma_{xy} \sin \theta \cos \theta \tag{5}$$

where $\theta$ is the angle measured from the $x$ axis [Fig. 3(c)].

RESULTS

Macroscale anchorage behavior

The strain development of the 30 m girder’s bottom along the span is shown in Fig. 5. Owing to the presence of camber, all strains were negative. A gradual increase in girder strain was observed until the CFRP termination point was reached, beyond which an abrupt strain change was noticed (i.e., net strain increase) because of the post-tensioned CFRP strip. This net strain increase can result in damage of the anchorage concrete when cracking takes place.

Mesoscale anchorage behavior

Figure 6(a) shows the initiation and propagation of mesoscale damage in the anchorage zone concrete (the simulation time indicates when the concrete damage initiated and propagated). The effect of post-tensioning (i.e., force applied to the anchor bolt) was insignificant until 0.59 sec; however, the change of element velocity was
noticed at 0.60 sec when the mesoscale damage nucleated. The stresses associated with these element responses are illustrated in Fig. 6(b). At a simulation time of 0.63 sec, the stress reached a maximum value of 3.4 MPa, which was close to the modulus of rupture suggested by ACI-318 (ACI 2014).

**Microscale anchorage behavior**

Figure 7(a) exhibits the damage development of the inclusion. The damage initiated at 100 microstrains and plateaued out up to 640 microstrains. This fact indicates that microscale damage rapidly increases when the NSM CFRP is post-tensioned. The variation of radial and circumferential stresses is provided in Fig. 7(b). The predicted radial stresses were gradually reduced as an angle increased, and were influenced by the degree of post-tensioning levels (20%$f_{tu}$ to 60%$f_{tu}$). The pattern of the circumferential stresses was opposite to that of the radial stresses.

**SUMMARY AND CONCLUSIONS**

This paper has discussed a multiscale modeling approach for the anchorage region of prestressed concrete bridge girders strengthened with post-tensioned NSM CFRP strips. Micro- to macroscale models were formulated to examine the damage initiation and propagation of the anchorage concrete when the CFRP was post-tensioned, based on finite element, discrete-object physics, and analytical methods. The macroscale strain response indicated that the NSM CFRP resulted in local tension near the anchorage. The nucleation of damage was sudden from a mesoscale standpoint, which was related to the interaction between the concrete and the anchor bolt. The interfacial transition zone was the place where microscale damage occurred, and its stress development was controlled by the post-tensioning level of the NSM CFRP.
ACKNOWLEDGMENTS

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DEFORMABILITY OF PRETENSIONED PC BEAMS WITH BFRP REINFORCEMENT

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ABSTRACT

The non-corrosive nature and high strength of fiber reinforced polymer (FRP) reinforcement makes it attractive for use in concrete applications. The main problem for using FRP reinforcement is its higher deformability especially for glass and basalt FRP. One of the possible approaches for reducing the deformations is via prestressing the reinforcement. This paper is a continuation of a previous research and presents experimental results of five large scale concrete beams reinforced to different levels of prestressing of the reinforcement. Four of the beams are reinforced with Basalt fibre reinforced polymer (BFRP) 10mm diameter bars. Test results showed that prestressing above 30% from the ultimate load for BFRP reinforcement results in similar or smaller deformation capacity of BFRP reinforced beam in comparison with steel reinforced for all levels of loading. The BFRP Beam with 30% and 40% of prestressing have higher capacity than the steel reinforced beam when a Serviceability Limit State (SLS) criterion is applied.

KEYWORDS

Large scale samples, RC beams, BFRP, Prestressing, Pre-tensioning,

INTRODUCTION

BFRP bars have emerged as an alternative to steel reinforcement due to their non-magnetic properties, non-corrosive nature, correspondingly lower life cycle cost and reduced CO₂ emissions. However, due to relatively a low elastic modulus, the main issue for using BFRP reinforcement is the high deformability, which can be controlled by prestressing the reinforcement. Several design codes have been proposed on carbon FRP (CFRP), glass FRP (GFRP) and aramid FRP (AFRP) bars for prestressed structures, namely in USA (ACI 440.4R.04, 2011), Canada (CAN/CSA S06-06, 2006), and Japan (JSCE CES23, 1997).

The aim of the paper is to analyse the experimental results reflecting the behaviour of BFRP reinforced beams with different level of prestressing and to compare them with a steel reinforced beams.

BACKGROUND

The use of FRP in reinforced concrete (RC) for anti-corrosion purposes is expected to find applications in structures in or near the ground, marine environments and in thin structural elements. In the early 90’s very intensive initial research and development on applications of FRP widely taking place in many countries, especially in North America, Europe and China. In 1988 the first application of FRP tendons using external prestressing for the pedestrian bridge in a Berlin park was constructed (Zia et al., 1997). In the late 80’s AFRP prestressing bars were used in the Netherlands to reinforce some of the posts of a noise barrier along a highway. This application was made because of the aggressive environment on those posts due to exposure to deicing salts and exhaust gasses of cars (Taerwe, 1993). In 1996 the Aberfeldy Footbridge in Scotland was constructed, it was the world’s first major advanced composite footbridge and remains the longest span advanced composite bridge in the world (Skinner, 2009).

Extensive research has shown that BFRP bars perform similarly to GFRP bars. RC beam with BFRP bars achieved short term strengths that are consistent with the relevant properties of the constituent materials (Patnaik et. al. 2010). Jonsson (2011) investigated RC beams pre-tensioned with BFRP tendons, he stated that the stiffness and bearing capacity of the BFRP beams significantly higher than un-prestressed beam. Pearson and
Donchev (2012) experimental results on post-tensioning BFRP RC beams with different levels of prestress showed that a significant reduction in the deformations can be achieved by applying an appropriate level of prestress. Guðmundsson (2012) investigated the moment capacity of four BFRP prestressed RC beams and commented that BFRP prestressed beams without shear reinforcement were weak in shear even if the shear span to depth (a/d) ratio was high. Revathy and Sriraman (2014) stated that the GFRP strengthened prestressed concrete (PSC) beam increased its load carrying capacity by 89 % and ductility by 90% over the control beam. Mirshekari et. al. (2015) investigated six large scale concrete beams reinforced with 6mm BFRP & steel bars and prestressed to a different degree. They pointed out that the prestressing of BFRP rebar had a significant effect on the deformability of the beams as the increasing level of prestress leads to a delay in the development of the initial cracks. The obtained Serviceability Limit State (SLS) level of capacity for prestressing at and above 40% is higher than for steel reinforced beam.

In parallel with excellent benefits from prestressing of BFRP reinforcement few disadvantages have to be considered. One of them is the reducing of the prestressing force during the process of the production of the elements (short term) and during the span of their life (long term). The estimation of the magnitude of the losses of the prestress is important factor for considering the behaviour of the elements. As indicated in (e.g. ISIS Canada 2006; Mirshekari et. al. 2016) the characteristics of the losses of prestress for FRP reinforcement are generally different from losses for steel reinforcement.

Another critical factor for consideration is the danger from creep rupture effects at high level of prestressing. From the extensive experimental work conducted in Kingston University and Reykjavík University (Jonsson 2011; Person and Donchev 2012; Guðmundsson 2012; Mirshekari et. al. 2015) the creep rupture effects have not been monitored and for this reason the creep rupture effect has been excluded from discussion in this paper.

The presented work is focused on investigating the effects of different levels of prestressing on internal BFRP and steel reinforcement on large scale RC beams. Mirshekari et al. (2015) investigated influence of the degree of prestressing on the behaviour of BFRP reinforced beams, this paper presents a second phase of this project.

METHODOLOGY

Description of Test Beams

Five RC beams with a total length of 2400 mm each are subjected of analysis in this publication. Four samples had BFRP reinforcement with a different degree of initial prestressing: 0%, 20% (16 kN), 30% (24kN) and 40% (31 kN) of the design ultimate tensile load (UTL) of BFRP reinforcement.

<table>
<thead>
<tr>
<th>Description</th>
<th>Key/Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Un prestressed Steel</td>
<td>S0</td>
</tr>
<tr>
<td>Un prestressed BFRP</td>
<td>B0</td>
</tr>
<tr>
<td>20% pretensioned BFRP (16 kN)</td>
<td>B20</td>
</tr>
<tr>
<td>30% pretensioned BFRP (24 kN)</td>
<td>B30</td>
</tr>
<tr>
<td>40% pretensioned BFRP (31 kN)</td>
<td>B40</td>
</tr>
</tbody>
</table>

The beams were designed to be tested over a span of 2000 mm and supported at 200 mm distance from the specimens’ ends. A load spreader beam allowed the application of loads at two points separated by a distance of 600 mm. The load was applied symmetrically about the mid-span of each test beam, resulting in four point testing. The diameter of BFRP reinforcement was 10mm for all beams and it was supplied by Magma Tech Ltd. The high yield steel was 10 mm in diameter used in S0 Control steel beam. To prevent shear failure 6 mm diameter mild steel links were placed at 100 mm intervals in shear spans of the beams. A 25 mm cover for main longitudinal reinforcement was provided. Steel tubes attached to the ends of the BFRP bars were used to prestress them and were left to act as additional anchorage throughout the testing to prevent premature anchorage failure. The arrangement of BFRP bar reinforcement and typical cross sectional details of beams are shown in Figure 1.
Figure 1 Experimental setup and cross-sectional details of beams tested (unit: mm)

Prestressing Set-up and Instrumentation

The prestressing force was applied via a pre-tensioning technique with a prestressing equipment as shown in Figure 2a. The levels of prestress before and during the process of casting, the change in prestress during releasing from prestressing devices and the development of internal stresses during the process of loading were monitored. The applied prestressing force was measured via load cells (Novatech F207 – 100 kN) for each of prestressed beams and the resulting strains measured via strain gauges for each of the bars during the whole process of prestressing. The prestressing force was checked before and after the casting of the concrete mix. Concrete was supplied by ready mix concrete manufacturer and all five beams were cast at the same time and from the same concrete mixture, to make them sufficiently similar to ensure a good level of comparability. The beams from this series were aimed to have concrete strength designed as C35/45 concrete mix. After 28 days the anchors are released and the prestress force is transferred to the beams.

Seven linear variable differential transducers (LVDTs) were attached on the beams as shown in Figure 1 and the strain gauges were placed on the tensile reinforcing bars at mid-span, quarter-span and close to anchorage position, additional strain gauge were attached to the steel reinforcing bars in the compression zone at mid-span position. Concrete strain gauges were also attached to the concrete surface at the top and bottom sides of the beams at mid-span. Loading was applied via two equal point loads (Figure 2b) at 2 kN load steps as total load with rate of application of 0.02 kN/s. At each step the load was kept for five minutes. During the process of testing the vertical displacements at mid-span, quarter spans, supports and two ends were measured. Crack appearance and their propagation were recorded.

Figure 2 Connection of the reinforcing BFRP bars to the threaded bars for prestressing (a) and test setup (b)
RESULT AND ANALYSIS

The results for the mid-span load-deflection curve are given in Figure 3. The comparison between un prestressed BFRP and steel beams indicates that the un prestressed BFRP beam (B0) is more deformable than steel beam (S0) as expected. When comparing the difference between un prestressed and prestressed BFRP beams, it shows that 20% BFRP prestressed beam (B20) has higher stiffness than B0 for whole interval of testing, which is more visible for loading above 6kN load. Furthermore, for level of loading between 10 and 48kN the difference in the deflections is almost the same with the maximum value of about 10mm. The behaviour 30% BFRP prestressed beam (B30) is closer to S0 with exceeding stiffness of the beam for predominant part of the testing interval with small exception at about 30kN. 40% kN BFRP prestressed beam (B40) has a significantly higher stiffness than S0 and much higher ultimate capacity. For levels of displacement above 10mm al load/deflection curves for all BFRP beams are almost parallel.

The failure loads for test beams are shown in Figure 4, when the load on the beam was not possible to be increased (maximum achievable force). Failure load for all BFRP beams is higher than the steel beam. The ultimate capacity of the prestressed BFRP reinforced beams appears to be significantly higher than for the un prestressed one. This effect probably is due to the reduced deformability of prestressed beams and is connected with a better distribution of stresses over the height of the section. The obtained experimental results from the BFRP reinforced beams with different level of prestressing show that the increase in the capacity is gradual and proportional to the increase of the level of prestressing. The difference in the capacity becomes smaller with the increasing level of prestressing (Figure 4).

![Figure 3 Mid-span deflection results for all beams at various level of load](image_url)

![Figure 4 Failure loads for all beams](image_url)
Comparing the deflection at ultimate load for steel reinforced beam (38kN) the BFRP beams with all level of prestressing show significantly lower deflections than B0. The reduction of deflection of unprestressed BFRP beam (B0) is 28% for B20, 55% for B30 and 76% for B40. The deflection for unprestressed steel beam (S0) is 33% less than B0 and 4% higher than B20, 40% of B30 and 68% of B40 (Figure 5).

![Figure 5 Deflection at ultimate load for steel reinforced beam (38 kN)](image)

Figure 5 Deflection at ultimate load for steel reinforced beam (38 kN)

Figure 6 shows the maximum load achieved at SLS limit of deflections, adopted as span divided by 360 according to BSI (2005). As it is visible in the chart BFRP samples with 30% (B30) and 40% (B40) of prestressing have higher capacity than the steel reinforced sample (S0) when a SLS criterion is applied.

![Figure 6 Load at 5.5mm deflection (SLS limit)](image)

Figure 6 Load at 5.5mm deflection (SLS limit)

Typical development of strain with load is as presented by the load-strain curve in Figure 7.

![Figure 7 Load strain curves for all beams at mid-span](image)

Figure 7 Load strain curves for all beams at mid-span
CONCLUSION

The comparison was conducted on basis of the same number of main longitudinal reinforcing bars. Based on the test results, the following conclusions can be drawn:

- Prestressing above 30% for BFRP reinforcement results in similar or smaller deformation capacity of BFRP reinforced beam in comparison with steel reinforced for all levels of loading.
- The ultimate capacity of all BFRP reinforced beams is higher than the capacity of steel reinforced beam and increasing with level of prestressing. It reaches 73% difference between control beam (S0) and 40% BFRP prestressed beam (B40).
- For SLS limit of Span/360, 30% BFRP prestressed beam (B30) and 40% BFRP prestressed beam (B40) have lower deformability than the steel reinforced beam.
- The developing strain follows in general the development of the different strains for all levels of prestressing.

Further research is needed to examine and clarify a range of unknown factors connected with the behaviour of prestressed concrete (PC) elements using FRP rebars such as losses of prestress, the behaviour at different levels of prestressing, partial prestressing and others.

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FATIGUE REPAIR OF METALLIC STRUCTURES WITH A THERMALLY-ACTIVATED SHAPE MEMORY ALLOY (SMA)/ CARBON FIBER REINFORCED POLYMER (CFRP) PATCH

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ABSTRACT

This paper introduces an approach of applying prestress/compressive stress to fatigue sensitive details of steel members using shape memory alloys (SMA) and carbon fiber reinforced polymer (CFRP) composites. The effectiveness of this approach in improving the fatigue behavior of steel members is also demonstrated. A total of 27 single edge-notched steel specimens were repaired using one of four different reinforcement configurations. The repaired specimens were tested under tension-tension fatigue loading at three different stress ranges up to failure. Test results showed that the average fatigue life of specimens that were reinforced with SMA/CFRP patches was 26 times that of the unreinforced specimens at a stress range of 155 MPa. At an applied stress range of 217 MPa the average fatigue life of the retrofitted specimens was 15 times that of the unreinforced specimens. The results indicate that the use of SMA/CFRP composites is a promising approach to extend the lives of metallic structures that are susceptible to fatigue-induced cracking.

KEYWORDS

Fatigue, repair of steel structures, shape memory alloys, CFRP.

INTRODUCTION

The use of CFRP patches has been recently demonstrated as an effective technique to improve the fatigue resistance of metallic structures (Jones and Civjan 2003; Yu et al. 2013). Using prestressed CFRP to strengthen fatigue sensitive and fatigue damaged structures is more effective than using non-prestressed patches (Taljsten et al. 2009; Huawen et al. 2010). The current technique for prestressing externally bonded FRP plates requires hydraulic jacks or heavy fixtures to apply the prestress and anchor the FRP. This strengthening method is practical for rehabilitation of individual members in a laboratory environment. However, its application may be limited on in-service structures due to accessibility constraints.

Prestrained SMA materials exhibit a unique thermomechanical response through which, when they are heated, they return to their un-prestrained geometries. By restraining this transformation, prestrained SMAs can be used to apply recovery forces, or prestress forces, to structural elements. This property of SMA materials has been exploited to develop composites that are able to actively tune their mechanical response (Shimamoto et al. 2004; Bollas et al. 2007). Similarly, El-Tahan and Dawood (2015) developed a thermally-activated SMA/FRP patch that can be anchored to structural members to apply prestressing forces through heating without the need for heavy equipment. In these patches nickel-titanium-niobium (NiTiNb) SMA wires are used to apply a prestressing force and a CFRP overlay is used to bridge the crack and protect the SMA wires. This research investigates the fatigue performance of steel members repaired using this newly developed SMA/CFRP patch. Through an experimental investigation, this research quantified the fatigue life enhancement that could be achieved using the proposed SMA/FRP patch.

DESCRIPTION OF THE SMA/FRP PATCH AND TEST SPECIMENS

The composite patch consists of martensitic SMA wires and high-modulus (HM) CFRP. The NiTiNb SMA wires used in this research were manufactured by Intrinsic Devices Inc. The wires were provided with a recoverable prestrain of 0.05 mm/mm and a grit blasted surface. These wires have a wide thermal hysteresis. The reported
austenite finish temperature, \( A_f \), and martensite start temperature, \( M_s \), of the wires are 160 and -50°C respectively. This ensures that the material will retain the recovery stress when the temperature decreases after the activation is completed. The temperature-recovery stress relationship, shown in Figure 1(a) was obtained from a single wire test. The wire was fixed between two grips connected to a load cell, and then heated to record the temperature-recovery force relationship. The wire was initially at room temperature 25°C. After heating the wire up to 160°C, the measured recovery stress in the wire was 400 MPa. When the wire was cooled to room temperature, the recovery stress remained constant as shown in the figure. The theoretical temperature-recovery stress relationship below room temperature is plotted as a dashed line in the figure.

In order to attach the SMA wires to the surface of steel members, the wires were embedded into two layers of dry HM carbon fiber at two ends as shown in Figure 1(b). Each patch included 46 SMA wires of 0.78 mm diameter \( \times \) 230 mm length. The spacing between the wires was equal to the wire diameter and the wires were embedded 76 mm into each CFRP patch. The carbon fabric was saturated with Araldite 420 resin and sandwiched between two plastic molds. The patch was left to cure for 7 days. The SMA wires in the middle segment were intentionally exposed to facilitate the heating activation after being attached to the steel surface.

Figure 1 a) Temperature and recovery stress relationship of the NiTiNb SMA material; b) SMA wires embedded into CFRP with exposed portion for heating activation

Figure 2 illustrates the procedures of using the thermally-activated system to strengthen a steel plate with a fatigue-sensitive detail. Figure 2(a) shows the 914 mm long, 102 mm wide, and 6.4 mm thick bare steel plate to be strengthened. A 6.35 mm deep, 60 degree edge-notch was generated at the center of one edge to induce a stress concentration. The SMA/CFRP patches were bonded to both sides of the steel plate using a two-part structural epoxy as shown in Figure 2(b). The SMA wires were activated after the epoxy cured for 7 days. As illustrated in Figure 2(c), the SMA/CFRP patches on both sides of the specimen were thermally insulated with glass fiber insulation and aluminum foil to provide a cool anchorage region and prevent the SMA wires from pulling out of the CFRP patch. Two strain gauges were installed close to the notch root on both sides of the steel substrate, and insulated with glass fiber, to record the strain change in the steel substrate during activation. Two thermocouples were placed under the SMA wires to record the temperature change. Subsequently, the exposed SMA wires were gradually heated up to 120°C using a digitally controlled forced-air heat gun and left to cool down to room temperature. The same procedure was repeated on both sides of the specimen. The SMA wires applied compressive stress to the steel substrate after the heating process. The results recorded by the thermocouples and strain gauges are discussed in the following sections. After activation the exposed SMA wires were covered with a structural epoxy as shown in Figure 2(d) to protect the activated wires and provide a flat surface on which to bond the CFRP overlay. Finally, two layers of unidirectional HM carbon fiber (356 mm \( \times \) 76 mm) were saturated with Araldite resin and overlaid on top of the SMA/CFRP patch using a hand lay-up technique. Figure 2(e) shows the final specimen.

Figure 2 Illustration of the procedure of reinforcing steel plate using the thermally-activated system
The prestressed SMA wires extended the fatigue life of the single edge-notched specimen primarily by reducing the stress ratio, which correspondingly reduced the effective stress intensity factor range at the crack tip. The measured modulus of the activated SMA wires is 70 GPa (data from the wire test). The ratio of the stiffness of the prestressed SMA wires to that of the steel substrate is less than 2%. As a result the stress range reduction achieved by the SMA wires was minimal. On the other hand, the prestressed SMA wires applied compressive stresses to the steel, which reduced the stress ratio. Figure 3 shows the relationship between the compressive stress in the steel and the temperature in the SMA wires during the heating process corresponding to step (c) in Figure 2. The strain in the steel close to the notch root was recorded using strain gauges. The temperature was monitored using two thermocouples. As shown in the figure, after heating the SMA wires on the first side of the sample to 120°C, an average compressive stress of 7 MPa was generated in the steel. By heating up the wires on the other side, the average compressive stress reached a peak value of 19 MPa. The resulting average compressive stress in the steel after cooling to room temperature was 17 MPa. The other specimens in the SMA and SMA/CFRP groups exhibited a similar trend with a resulting average compressive stress in the steel of approximately 17 MPa after the thermal activation of the SMA wires. Due to fabrication and specimen imperfections, the resulting stresses on both sides of the steel were not identical having a maximum mismatch of 30% based on the measured strains shown in Figure 3. The compressive stress provided by the SMA wires reduced the stress ratio in the steel, while the stress range in the steel remained unchanged.

**EXPERIMENTAL PROGRAM**

*Specimen description and test setup*

In order to evaluate the effectiveness of the SMA/CFRP system, four groups of coupons were prepared and tested under high-cycle, tension-tension fatigue loading. The tested groups included: (i) un-strengthened, plain steel control coupons; (ii) coupons patched with bonded CFRP only; (iii) coupons patched with prestressed NiTiNb SMA wires only; and (iv) coupons patched with SMA/CFRP composite. Three stress ranges, 217 MPa, 155 MPa and 93 MPa were considered. Each testing configuration was assigned a two-part designation. The first part indicates the specimen detail: plain steel control coupons (Steel), samples repaired with a CFRP patch (CFRP), samples repaired with SMA wires (SMA), and samples repaired with SMA wires overlaid with a CFRP patch (SMA/CFRP). The second part indicates the far-field stress range of the fatigue test in MPa. Table 1 summarizes all of the specimen configurations. Three repetitions were conducted for each configuration; each specimen has a unique third part serial number to distinguish between multiple repetitions of the same test configuration.

Three steel tension coupons and three Charpy test coupons were fabricated and tested according to ASTM E8 (2013) and ASTM E23 (2012) respectively. The average measured elastic modulus, yield stress, and room-temperature Charpy V-notch energy of the steel are 200 GPa, 400 MPa, and 67 J respectively. All the steel plates used in this research were fabricated from one batch of material to minimize inter-sample variability. The tensile strength, elongation and elastic modulus of the HM CFRP obtained from tension coupon tests according to ASTM D3039 (2014) are 345 MPa, 0.0025 mm/mm, and 138 GPa, respectively. The elastic modulus, ultimate strength and ultimate strain of the Araldite 420 resin obtained from coupon tests according to ASTM D638 (2010) are 3.05 GPa, 62 MPa and 0.022 mm/mm respectively.
Table 1 Test Matrix for all the fatigue tests

<table>
<thead>
<tr>
<th>Configuration Designation</th>
<th>Far-field stress range [MPa]</th>
<th>Stress range [MPa]/stress ratio after strengthening</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel-217</td>
<td>217</td>
<td>217/0.1</td>
<td></td>
</tr>
<tr>
<td>Steel-155</td>
<td>155</td>
<td>155/0.1</td>
<td></td>
</tr>
<tr>
<td>Steel-93</td>
<td>93</td>
<td>93/0.1</td>
<td></td>
</tr>
<tr>
<td>CFRP-217</td>
<td>217</td>
<td>187/0.1</td>
<td>Patched with CFRP only</td>
</tr>
<tr>
<td>CFRP-155</td>
<td>155</td>
<td>133/0.1</td>
<td></td>
</tr>
<tr>
<td>CFRP-93</td>
<td>93</td>
<td>80/0.1</td>
<td></td>
</tr>
<tr>
<td>SMA-155</td>
<td>155</td>
<td>155/0.0</td>
<td>Patched with SMA wires only</td>
</tr>
<tr>
<td>SMA/CFRP-217</td>
<td>217</td>
<td>133/0.0</td>
<td>Patched with SMA &amp; CFRP overlay</td>
</tr>
<tr>
<td>SMA/CFRP-155</td>
<td>155</td>
<td>187/0.03</td>
<td></td>
</tr>
</tbody>
</table>

The tests were carried out using a MTS servo-hydraulic testing frame with a capacity of 500 kN. All the specimens were tested under constant amplitude, sinusoidal, tension-tension fatigue loading with a load ratio of 0.1 and frequency of 10 Hz. To determine the crack length in the steel substrates at different numbers of fatigue cycles, the beach marking technique was employed (Yu et al. 2013). Figure 4 shows part of the loading history of a specimen and the fracture surface with beach marks to illustrate this technique. Half-amplitude loading cycles were conducted at predefined intervals, as shown in Figure 4(c). During these reduced amplitude loading packets the crack propagation rate decreased substantially generating a dark band on the fracture surface as shown in Figure 4(b). These bands, which were formed at known cycle counts were used to calculate the crack length for a given number of applied cycles. Since the half-amplitude cycles did not substantially propagate the crack, they were not considered in the total cycle count. The maximum and minimum applied loads were recorded every ten cycles to ensure consistency of the loading protocol. The beach marks were slightly asymmetric with respect to the mid-thickness plane which may have been due to the difference of the recovery forces of the SMA wires on both sides.

Figure 4(a) shows the crack front trace on the fracture faces of a single specimen after complete fracture. For convenience, the behavior of the coupons was compared at different stages throughout the loading corresponding to the instances of the reduced-amplitude beach marking cycles. At a given stage in the loading, the applied number of load cycles, N, was related to the beach marks by counting from the last beach mark and calculating the associated number of full-amplitude loading cycles. If the sample failed during the full-amplitude loading cycles, the number of cycles at failure was taken directly from the test frame controller (after subtracting the cumulative number of reduced-amplitude beach marking cycles). If failure occurred during the reduced-amplitude beach marking cycles, the number of cycles at failure was conservatively taken as the last number of full-amplitude cycles prior to failure. As illustrated in Figure 4(a), the crack length corresponding to the $i^{th}$ beach mark, $a_i$, in this research was measured from the root of the 60° edge-notch to the $i^{th}$ beach mark at the mid-thickness of the specimen.

Figure 4 Part of the loading history including the beach marking cycles

Results and discussion

All the specimens experienced abrupt failures due to the rupture of steel when the crack reached a critical length after a certain number of fatigue loading cycles. The patches that were attached to the surface of the steel either fractured at the location of the crack or debonded from the substrate with the rupture of the steel plate. Figure 5 plots the measured fatigue lives of the tested specimens against the stress range. The fatigue lives corresponding to the American Association of State and Highway Transportation Officials (AASHTO) and American Institute of...
Steel Construction (AISC) fatigue categories B’ and E fatigue details are indicated in the figure for reference. As a baseline, it can be seen that the unpatched control specimens had fatigue lives that fall below the AASHTO and AISC fatigue category E. Fatigue lives of all of the tested specimens are summarized in Figure 5. Specimens that were patched with CFRP only and tested at a far-field stress range of 93 MPa exhibited fatigue lives of more than 2 million cycles without any appreciable crack propagation, which is consistent with the constant amplitude fatigue threshold for a fatigue category B’ detail. For samples that were tested at a stress range of 155 MPa, installation of the CFRP patch increased the average fatigue life of the fatigue detail to 8.7 times that of the unpatched control samples, while for a stress range of 217 MPa the average fatigue life of the CFRP-patched samples was 3.0 times that of the unpatched samples. This is equivalent to upgrading the fatigue detail to an AASHTO/AISC category C detail.

The specimens that were retrofitted with only SMA wires bridging the crack (without a CFRP overlay) exhibited an average fatigue life that was 70% greater than that of the control group for a far-field stress range of 155 MPa. It can be seen that the influence of the prestressed SMA wires only was relatively minor compared to that of the CFRP patches. The samples that were retrofitted with the thermally-activated SMA/CFRP patches and tested at stress ranges of 155 MPa and 217 MPa exhibited average fatigue lives that were, respectively, 26 and 15 times those of the control group. As such, installation of the thermally-activated SMA/CFRP patch increased the fatigue lives of the specimens from below that of a category E detail to above that of a category B’ detail.

<table>
<thead>
<tr>
<th>Stress range [MPa]</th>
<th>Steel</th>
<th>CFRP</th>
<th>SMA</th>
<th>SMA/CFRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>93</td>
<td>390,947</td>
<td>2,000,000</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>155</td>
<td>49,868</td>
<td>396,000</td>
<td>73,495</td>
<td>1,151,907</td>
</tr>
<tr>
<td>217</td>
<td>15,297</td>
<td>37,000</td>
<td>N/A</td>
<td>274,846</td>
</tr>
<tr>
<td></td>
<td>15,500</td>
<td>86,000</td>
<td>243,463</td>
<td></td>
</tr>
</tbody>
</table>

* fatigue testing runout of 2,000,000 cycles

Figure 6 compares the crack growth (a-N) curves of the specimens that were patched with CFRP only to those of the specimens that were patched with the thermally-activated SMA/CFRP patches. Figure 6(a) shows the specimens that were tested with far-field stress range of 217 MPa. The fatigue life was substantially increased by the SMA/CFRP reinforcing system compared to the CFRP reinforced specimens. Further, among the specimens that were retrofitted with the SMA/CFRP patches even the specimen with the lowest fatigue life exceeded the life of the fatigue category B’ detail. The specimens reinforced with CFRP exhibited lower fatigue lives than AASHTO fatigue category C. Figure 6 b) shows the specimens tested at a far-field stress range of 155 MPa. The scatter of the fatigue lives may have been due to the combination of the inherent variability in crack initiation and differences between the recovery stresses generated by the SMA wires. However, the shortest fatigue life among these specimens still exceeded the fatigue life of an AASHTO category B’ detail.
CONCLUSIONS

This paper discusses the effectiveness of a newly-developed thermally-activated SMA/CFRP patch for extending the fatigue lives of fatigue-sensitive steel details. The approach exploits the unique thermomechanical properties of SMA wires to apply compressive stresses to the steel elements. Four groups of single edge-notched specimens were reinforced with different reinforcement configurations and tested under tension-tension fatigue loading up to failure at three different stress ranges. The results showed that the thermally-activated SMA/CFRP composite patches increased the average fatigue lives of edge-notched steel plates to 26 and 15 times those of the un-patched plates at 155 MPa and 217 MPa far-field stress ranges, respectively. This improvement was equivalent to upgrading the fatigue detail from worse than an AASHTO category E detail to better than a category B’ detail. In comparison, the specimens that were patched with CFRP only had average fatigue lives that were 3 and 8.7 times those of the unpatched samples at stress ranges of 155 and 217 MPa, respectively. The prestressed SMA wires alone prolonged the average fatigue life by 70% at the 155 MPa stress range. This indicates that the thermally-activated SMA/CFRP patching system is much more effective than the simple sum of the reinforcing effects of CFRP and prestressed SMA wires. It is evident that there is a synergistic effect between the CFRP and prestressed SMA wires in strengthening fatigue sensitive steel details. The results suggest that the thermally-activated SMA/CFRP patch is a promising system for improving the lives of fatigue-sensitive metallic members.

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REFERENCES

ABSTRACT

The application of Carbon Fibre Reinforced Polymers (CFRP) in strengthening and retrofitting of existing structures has emerged as a practical and efficient technique compared to the application of conventional materials (i.e. steel and concrete). In case of flexural strengthening of reinforced concrete (RC) beams, prestressed CFRP provides better performance at service conditions compared to non-prestressed CFRP. Recently, researchers have utilized the Iron-based Shape Memory Alloy (Fe-SMA) reinforcement (bars or strips) for strengthening applications of RC beams. The Fe-SMA reinforcement is a metallic alloys that can recover part of the induced tensile strain upon heating. If the pre-strained Fe-SMA reinforcement is restrained (i.e. anchored to the tension side of the beam), the heating will trigger the inherited shape memory recovery property to be initiated. However, because the SMA reinforcement is restrained, a prestressing force will be developed instead. This paper presents a numerical comparison between large-scale RC beams strengthened with prestressed Near-Surface Mounted (NSM) CFRP rods and self-prestressing NSM Fe-SMA strips with a comparable amount of prestressing forces. Both techniques show a similar performance at the service load conditions. While the beams strengthened with prestressed NSM CFRP rods show strength-ductility trade-off, the beams strengthened with NSM Fe-SMA strips exhibited more ductile behaviour, this is attributed to the yielding nature of the Fe-SMA material.

KEYWORDS

Carbon Fibre Reinforced Polymer, ductility, finite element, iron-based shape memory alloys, service, strengthening, ultimate.

INTRODUCTION

Strengthening RC structures with externally bonded (EB) or near-surface mounted (NSM) CFRP reinforcement (sheets, rods, strips) has become one of the major research topics in structural engineering. This is driven by the need to retrofit/strengthen deteriorated infrastructures around the globe. Most of the CFRP-strengthening applications are passive techniques, where the CFRP reinforcement are applied to increase the ultimate capacity of the structures (El-Hacha and Rizkalla 2004). Passive strengthening techniques have minimal effect on the serviceability performance of the structure. To improve the performance of structures at service conditions, researchers introduced the prestressing CFRP as an active strengthening technique (El-Hacha et al. 2001, Wight et al. 2003, El-Hacha and Soudki 2013). In the case of RC beams, the CFRP material is prestressed against the RC beam in a way to counteract the applied loads, and consequently, improve the cracking load, yielding load, and the ultimate load of the beam. While the prestressing technique contributes significantly in improving the serviceability performance of RC beams, this comes at the cost of reducing the ductility. The more strain utilized for prestressing the more brittle the beams become (El-Hacha and Gaafar 2011). In addition to that, the application of prestressing force to the CFRP requires special end anchorage system and jacking tools. These are two main concerns that were the driving force behind the current research project that involves the utilization of the Iron-based Shape Memory Alloys (Fe-SMA) for flexural strengthening of RC beams.

The SMAs are metallic alloys that are mainly characterized by the ability to recover their original shape upon heating (Lagoudas 2008, Cismasiu 2010). The Fe-SMA is relatively inexpensive compared to the common shape memory alloys such as Nickel Titanium (NiTi), which makes it more suitable for large-scale structural engineering applications (Cladera, Weber et al. 2014, Czaderski, Shahverdi et al. 2014). Recently, Fe-SMA bars and strips for were used for flexural strengthening of small-scale RC beams using the Near Surface Mounted (NSM) strengthening technique (Rojob and El-Hacha (2016), Shahverdi, Czaderski et al. (2016)). In the NSM technique, the strengthening materials is embedded in a pre-cut groove at the tension side of the beam. The initial strain was
applied to the Fe-SMA materials before being anchored to the ends of the beam. The Fe-SMA material was placed inside the groove, then heated to the activation temperature which triggered the Shape Memory Effect (SME) and caused the prestressing force to be developed in the Fe-SMA bars or strips (Rojob and El-Hacha 2016).

The Fe-SMA reinforcement have a great potential in the field of cast-in-place prestressed concrete as an alternative to the high-strength steel cables or CFRP tendons. However, the current available Fe-SMA reinforcement are not comparable in strength to the steel cables nor to the CFRP tendons. In spite of the advantages of the strengthening system using the self-prestressing NSM Fe-SMA strip mentioned above, it is a steel-based material which makes it susceptible to corrosion. Although materials such as Chrome and Nickel could be added to develop a high corrosive-resistant SMA reinforcement, there is a need to study the long-term behaviour of this material under different environmental conditions.

This paper is part of an ongoing research project that involves strengthening of large-scale RC beams using NSM Fe-SMA strips. The beams were built to match other beams strengthened with prestressed CFRP rod and tested by El-Hacha and Gaafar 2011). The current paper presents a numerical comparison between the beams strengthened with NSM CFRP rods and similar beams strengthened with NSM Fe-SMA strips as part of the ongoing project. No experimental results are available at the time this paper is being prepared.

DETAILS OF THE RC BEAMS

Geometry

Figure 1 shows the details of a typical RC beam used in this study. The beam was reinforced with 3-15M bars in tension and 2-10M bars in compression with total cross-sectional area of 600 mm² and 200 mm², respectively. 10M bars were also used to form the shear stirrups. The spacing was 300 mm in the region of zero shear and 200 mm elsewhere. The beams were tested on four-point bending set-up. The NSM technique was used for strengthening using both CFRP rods and Fe-SMA strips.

Material Characteristics

The concrete compressive strength was 40 MPa and the steel yield strength was 475 MPa. The CFRP rods are 9 mm in diameter possessing linear elastic behaviour up to failure, with a modulus of elasticity of 130 GPa and an ultimate tensile strength and strain of 2167 MPa and 1.66% respectively (El-Hacha and Gaafar 2011). The Fe-SMA strips are non-linear material as shown in the stress-strain curve in Figure 2. Figure 3 shows the shape memory effect process of the Fe-SMA strips. The induced initial strain for the strip was 4%. During the heating and cooling process, the ends of the strips are restrained. The reduction in the stress at the beginning of the heating process was due to the thermal expansion effect, the stress started to pick up at temperature of 65 °C. During the cooling process, the stress increased due to the removal of the thermal expansion effect. The difference between the stress at the end of the cycles and the stress at the beginning of the cycles is the total recovery stress. The recovery stress of this Fe-SMA strips is about 260 MPa. The total area of the Fe-SMA strips was chosen such that the amount of prestressing force will match the prestressing forces induced in the CFRP rods as reported by El-Hacha and Gaafar (2011). Table 1 shows the beams to be modeled and the details of the strengthening materials.

DESCRIPTION OF FE MODEL

A 3D finite element model of the RC beams was developed using ABAQUS. Only quarter of the beams was modelled making use of two planes of symmetry. To insure the symmetry, boundary conditions were imposed at the symmetric surfaces as shown in Figure 4. The mesh sensitivity analysis was conducted to make sure that the
results are mesh independent. The concrete, and the steel plates used at the loading points and at the supports were modelled using linear solid elements, while the internal steel, CFRP and Fe-SMA reinforcement were modelled using linear truss element. The interaction between the reinforcements and the concrete were imposed through the embedded region option, were the translation of the reinforcements nodes are restricted by the translations of the concrete nodes. The beams were loaded in a displacement-controlled mode to get the full load-deflection response.

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>CFRP</th>
<th>CFRP-2</th>
<th>SMA-1</th>
<th>SMA-2</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strengthening Reinforcement</td>
<td>Shape</td>
<td>Rod</td>
<td>Strip</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dimensions (mm)</td>
<td>9 (Diameter)</td>
<td>1.5×10</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td># of units</td>
<td>1</td>
<td>7</td>
<td>14</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Prestressing force (kN)</td>
<td>28</td>
<td>56</td>
<td>27.3</td>
<td>54.6</td>
<td></td>
</tr>
</tbody>
</table>

Material Models

Concrete

The concrete behaviour in compression was modelled using the stress-strain model developed by Carreira and Chu (1985) as presented in Equations 1 to 3 and shown in Figure 5.

![Stress-strain curve of Fe-SMA strip](after Re-fer, 2015)

![SME process](after Re-fer, 2015)

![Configuration of the FE model for Beam C](loading plate, support, Uy=0, Uz=0, Ux=0)
\[ f_c = f_c' \left( \frac{Bx}{8 - 1 + x} \right) \]  \hspace{1cm} (1)

\[ B = \left( \frac{1}{1 - f_c'/\varepsilon_0} \right); \text{ and } x = \frac{\varepsilon_c}{\varepsilon_o} ; \varepsilon_o = 0.0029 \]  \hspace{1cm} (2)

\[ E_{it} = 3320\sqrt{f_c'} + 6900 \text{ MPa} \]  \hspace{1cm} (3)

where, \( f_c \) is the concrete stress in MPa, \( \varepsilon_c \) is the concrete strain, and \( E_{it} \) is the initial tangent modulus of the concrete in MPa.

The concrete in tension was modeled using the stress-displacement model developed by Coronado and Lopez (2006) as shown in Figure 6. The concrete tensile strength and fracture energy were calculated based on the CEB-FIP (1993) model as shown in Equations 4 and 5, respectively.

\[ f_t = 1.4 \left( \frac{f_c' - 10}{8} \right)^{2/3} = 3.04 \text{ MPa} \]  \hspace{1cm} (4)

\[ G_F = \left( 0.0469d_a^2 - 0.5d_a + 26 \right) \left( \frac{f_c'}{10} \right)^{0.7} = 0.0917 \text{ N/mm} \]  \hspace{1cm} (5)

where \( f_t \) is the concrete tensile strength, \( G_F \) is the fracure energy of concrete, \( f_c' \) is the concrete compressive strength, and \( d_a \) is the maximum aggregate size in mm.

**Steel**

The actual stress-strain curve of steel as reported by El-Hacha and Gaafar (2011) was simplified into a tri-linear curve as shown in Figure 7.
**Strengthening Materials**

The CFRP rod was modelled as linear elastic up to failure. The Fe-SMA strip was modelled using the actual nonlinear stress-strain curve as shown in Figure 2. The prestressing force was modelled as a predefined value of stress in the second step of the analysis.

**RESULTS AND DISCUSSIONS**

**Validation of the FE model:**

To validate the FE model, the predicted load-midspan deflection curves of beams B-C, CFRP-1, were compared to the experimental results reported by El-Hacha and Gaafar (2011) as shown in Figure 8. The results show a good agreement between the numerical and experimental results. The FE model results are a bit stiffer than the experimental results especially for beam CFRP-1. This is attributed to the fact that in the actual test results, there was a slip between the CFRP rod and the epoxy adhesive filling the groove and the surrounding concrete as reported by El-Hacha and Gaafar (2011) which was not accounted for in the FE model. As the model is verified, it will be used in the following section to compare between the beams strengthened with prestressed NSM CFRP rods and the beams strengthened with NSM Fe-SMA strips.

**Comparison between the CFRP and Fe-SMA**

Figure 8 FE results versus experimental results for beams C and CFRP-1

Figure 9 FE results of the load deflection curves for beams strengthened with CFRP and Fe-SMA

Figure 9 shows the load-midspan deflection curves of the beams strengthened with NSM Fe-SMA strips and those strengthened with NSM CFRP rod. Beam CFRP-1 and beam SMA-1 have a prestressing force of 28 kN and 27.3 kN, respectively. Both beams have a comparable behaviour at service load (i.e. cracking and yielding loads) as presented in Table 2. Beam CFRP-1 has a higher ultimate load compared to beam SMA-1, however, this comes at the cost of reducing the ductility of the beam and results in a sudden failure by rupture of the CFRP rod at a relatively small deflection. On the contrary, beam SMA-1 shows a 60% increase in the ultimate load over the control beam (only 10% less than beam CFRP-1) without compromising the ductility. In fact, the failure pattern of beam SMA-1 is not different than the behaviour of an under-reinforced concrete beam that starts with the yielding of the reinforcements (i.e. the internal steel rebars and the Fe-SMA strips) before the crushing of the concrete. This is due to the yielding nature of the Fe-SMA material that allows the RC beam to deflect significantly before the crushing of the concrete.

The same scenario can be observed when comparing beam CFRP-2 and beam SMA-2. Both beams with prestressing forces of 56 kN and 54.6 kN, respectively, have a comparable behaviour at the service load. While beam CFRP-2 experienced more brittle behaviour than beam CFRP-1, beam SMA-2 had a ductile failure comparable to beam SMA-1 and to the control beam. Because the axial stiffness of the Fe-SMA strips in beam SMA-2 is higher that the axial stiffness of the CFRP rod in beam CFRP-2, this allows beam SMA-2 to have a higher ultimate capacity. In addition to the ductile nature of the beams strengthened with Fe-SMA strips, the other advantage of the self-prestressing NSM Fe-SMA strengthening technique over the prestressed NSM CFRP strengthening is the easiness of applying the prestressing force. The prestressing force in the Fe-SMA strips can be applied by heating the Fe-SMA material through electric current (or other heating procedures) just above the activation temperature (150 °C for the Fe-SMA strip used herein); a process that is done after the curing of the grouting material while no anchorage or jacking tools are needed.
Table 2 Summary of FE model results

<table>
<thead>
<tr>
<th>Variable</th>
<th>B-C</th>
<th>CFRP-1</th>
<th>SMA-1</th>
<th>CFRP-2</th>
<th>SMA-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cracking Load: $P_{cr}$ (kN)</td>
<td>32</td>
<td>52.6</td>
<td>53.6</td>
<td>65.5</td>
<td>65</td>
</tr>
<tr>
<td>Deflection at Cracking: $\Delta_{cr}$ (mm)</td>
<td>3.34</td>
<td>4.51</td>
<td>4.17</td>
<td>4.26</td>
<td>4.02</td>
</tr>
<tr>
<td>Yielding Load: $P_{y}$ (kN)</td>
<td>82.4</td>
<td>119</td>
<td>120</td>
<td>136</td>
<td>153</td>
</tr>
<tr>
<td>Deflection at Yielding: $\Delta_{y}$ (mm)</td>
<td>22.5</td>
<td>26.6</td>
<td>24.3</td>
<td>24.3</td>
<td>25.8</td>
</tr>
<tr>
<td>Ultimate Load: $P_{u}$ (kN)</td>
<td>90.7</td>
<td>160.9</td>
<td>145</td>
<td>162.8</td>
<td>188.9</td>
</tr>
<tr>
<td>Deflection at Ultimate: $\Delta_{u}$ (mm)</td>
<td>157.8</td>
<td>91.88</td>
<td>163.5</td>
<td>53</td>
<td>139.6</td>
</tr>
<tr>
<td>Energy Dissipated (kN.mm)*</td>
<td>13113</td>
<td>11537</td>
<td>21601</td>
<td>6840</td>
<td>23466</td>
</tr>
<tr>
<td>Ductility: $\Delta_{u} / \Delta_{y}$</td>
<td>7.01</td>
<td>3.45</td>
<td>6.72</td>
<td>2.18</td>
<td>5.4</td>
</tr>
</tbody>
</table>

* The energy dissipated was calculated as the area under the load deflection curve up to the ultimate load.

CONCLUSIONS

This paper has presented a comparison between two active strengthening techniques, prestressed NSM CFRP rod and self-prestressed NSM Fe-SMA strips with comparable induced prestressing force on the RC beams. The Fe-SMA strips show a potential capability in strengthening RC beams for flexure while not compromising the ductility. Additionally, applying the prestressing force is much easier in the case of NSM Fe-SMA strips compared to the NSM CFRP rods due to the inherited shape memory effect phenomena which can be triggered by heating. More research is required to examine the corrosion-resistivity of this material, also a higher strength Fe-SMA is required if the material is to be used for cast-in-place prestressed concrete applications.

ACKNOWLEDGEMENT

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REFERENCES


FATIGUE LIFE IMPROVEMENT OF INITIALLY CRACKED STEEL PLATES USING CFRP/SMA COMPOSITES EMBEDDED WITH NI TI SHAPE-MEMORY ALLOY WIRES

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ABSTRACT

Fatigue is one of the most important failure mechanisms that affect steel structures during their design life. Therefore, fatigue problems affecting steel structures due to external effects (cyclic loading, dynamic loading, etc.) have become a major concern for structural engineers. Conventional methods for fatigue life improvement of steel structures include attaching external steel plates. This has many disadvantages and deficiencies, as the steel plates are large in size, heavy in weight and they use the same material (steel), which is also subject to corrosion and fatigue. Therefore, there is a need for new advanced methods for fatigue life enhancement of steel structures. This paper reports the development of an active hybrid CFRP/SMA composite made of unidirectional normal modulus carbon fibre reinforced polymer (CFRP) fabric embedded with pre-strained body temperature nickel titanium (NiTi) shape-memory alloy (SMA) wires. The study involved the fabrication of CFRP/SMA composites with different levels of pre-stressing capabilities. The composites were then attached using a strong structural epoxy adhesive to initially notched steel plates to study their effectiveness in improving the fatigue life of the steel plates. The experimental work included cyclic fatigue testing on un-strengthened steel plates, cyclic fatigue testing on steel plates strengthened with CFRP patches and cyclic fatigue testing on steel plates strengthened with CFRP/SMA patches. It was found that embedding pre-strained body temperature NiTi SMA wires inside the CFRP/SMA patches effectively increases the fatigue life of initially cracked steel plates.

KEYWORDS

CFRP, NiTi SMA, CFRP/SMA composites, fatigue, steel structures, strengthening, smart materials.

INTRODUCTION

Steel structures subjected to cyclic loading such as bridges and offshore platforms, are at risk of failure due to the presence of fatigue cracks created by fatigue. Therefore, it is very important that these fatigue cracks are repaired in an efficient and safe way to prevent any possibility of failure of the steel structure. Fatigue life enhancement is one of the most important parts of the repair and rehabilitation process in steel structures. In this paper, an advanced method using active smart composites is proposed for use in improving the fatigue life of steel structures subjected to cyclic loading. CFRP and SMAs with their unique properties and behaviour are very important materials, because of their great potential for application in civil engineering. Therefore, the combination of CFRP and SMAs will produce a smart active hybrid composite patch with many possible capabilities and applications in civil engineering. Active hybrid CFRP/SMA composites could be effectively used for improving the fatigue and creep properties of structures, or improving their damping capacity, or controlling their shape or vibration properties. Liu and Li (2006) presented an innovative repair method for strengthening damaged reinforced concrete (RC) beams using CFRP plates in combination with SMA wires. Wierschem and Andrawes (2010) studied experimentally and analytically the use of super-elastic SMA-FRP composites for the reinforcement of concrete structures. Zhang et al. (2006) presented the use of CFRP matrix composites embedded with SMA wires for vibration control under impact loading. Xu et al. (2003) investigated the development of shape-memory alloy/carbon fibre reinforced plastic hybrid composites for damage suppression at ambient temperature. In addition, Lee et al. (2004) proved that crack closure is greatly improved by the application of NiTi SMA- CFRP composites.
EXPERIMENTAL WORK

Fabrication of CFRP and CFRP/SMA Patches

Five types of patches were fabricated in the experimental work reported here. The first type was CFRP fabric patches with two 0° direction CFRP layers and without embedded SMA wires. This was given the name CFRP-2L. The second type was CFRP fabric patches with two 0° direction CFRP layers plus one 90° direction CFRP layer and without embedded SMA wires. This was given the name CFRP-(2+1)L. The third type was CFRP fabric patches with two 0° direction CFRP layers plus one 90° direction CFRP layer embedded with 15 non-pre-strained body temperature SMA wires. This was given the name CFRP-(2+1)L/15BT-SMA(NP). The fourth type was CFRP fabric patches with two 0° direction CFRP layers plus one 90° direction CFRP layer embedded with 9 pre-strained body temperature SMA wires. This was given the name CFRP-(2+1)L/9BT-SMA(P). The fifth type was CFRP fabric patches with two 0° direction CFRP layers plus one 90° direction CFRP layer embedded with 15 pre-strained body temperature SMA wires. This was given the name CFRP-(2+1)L/15BT-SMA(P). Two fabrication methods were adopted in this study to manufacture the five types of composites. The first fabrication method was designed to produce the first and second type of patch. The second fabrication method was designed to produce the third, fourth and fifth type of patch.

Fabrication Method 1

The fabrication process for the first type (CFRP-2L) and the second type (CFRP-(2+1)L) of patches was conducted following the procedure presented below and shown in Figure 1:

(a) Cut the CFRP fabric sheets to the required dimensions (50mm x 250mm). Next place the first bottom CFRP (0° ply) on a plastic sheet and then apply a uniform thin layer of epoxy adhesive on it. Then turn the CFRP ply to the other side and apply a uniform pressure using a roller to help the epoxy to be absorbed by the CFRP fabric;
(b) Place the saturated CFRP ply on a flat bottom steel plate and then apply layer of the epoxy adhesive on it;
(c) Place the middle layer of the CFRP (90° ply) fabric over the epoxy adhesive and then apply a uniform pressure on the CFRP fabric surface using a roller to help the epoxy adhesive to be absorbed by the CFRP fabric (This step is omitted for the first type (CFRP-2L);
(d) Apply a second layer of epoxy adhesive over the middle layer of the CFRP (90° ply) fabric and distribute the epoxy adhesive evenly along with the fibre direction (This step is omitted for the first type (CFRP-2L);
(e) Place the top layer of the CFRP (0° ply) fabric over the epoxy adhesive and then apply uniform pressure on the CFRP fabric surface using a roller to help the epoxy to be absorbed by the CFRP fabric. Two aluminium plates are attached to both sides of the CFRP fabric patch to provide a uniform thickness for the patch;
(f) Place the top flat steel plate over the top layer of the CFRP fabric. Press the top steel plate to remove the excess epoxy adhesive and to maintain a uniform epoxy adhesive thickness between the top CFRP layer and bottom CFRP layer. The patch is left to cure at room temperature for a minimum of seven days.

![Figure 1 Fabrication process for the first type (CFRP-2L) and the second type (CFRP-(2+1)L) of patch](image)

Fabrication Method 2

The fabrication process for the third type (CFRP-(2+1)L/15BT-SMA(NP)), the fourth type (CFRP-(2+1)L/9BT-SMA(P)) and the fifth type (CFRP-(2+1)L/15BT-SMA(P)) of patches was conducted following the procedure presented below and shown in Figure 2:
(a) Cut the SMA wires (1.00 mm in diameter) to the required length (approximately 400 mm), then sand-blast the SMA wires to increase their surface roughness for better bonding inside the CFRP/SMA patch. Then pre-strain the SMA wires to an approximate value of 6% using a special pre-straining device (This step of pre-straining is omitted for the third type of patch):
(b) Attach the pre-strained or non-pre-strained SMA wires on a special frame (called Frame 1). Frame 1 is used to restrain the pre-strained wires and prevent the loss of the pre-straining during the fabrication process. The frame helps to maintain uniform spacing between the SMA wires inside the CFRP/SMA patch;
(c) Cut the CFRP fabric sheets to the required dimensions (50mm x 250mm) and then place the first bottom CFRP (0° ply) on a plastic sheet and apply a uniform thin layer of epoxy adhesive to it. Then turn the CFRP ply over and apply uniform pressure using a roller to help the epoxy to be absorbed by the CFRP fabric;
(d) Place the saturated CFRP ply on a flat bottom steel plate and then apply layer of the epoxy adhesive to it;
(e) Place the special frame with the pre-strained or the non-pre-strained SMA wires over the epoxy adhesive and the bottom CFRP sheet. Add more epoxy to cover the SMA wires and smooth and level the epoxy surface;
(f) Place the middle layer of the CFRP (90° ply) fabric over the epoxy adhesive and the SMA wires, then apply uniform pressure on the CFRP fabric surface using a roller to help the epoxy adhesive to be absorbed by the CFRP fabric;
(g) Apply a second layer of epoxy adhesive over the middle layer of the CFRP (90° ply) fabric and distribute the epoxy adhesive evenly along with the fibre direction;
(h) Place the top layer of the CFRP (0° ply) fabric over the epoxy adhesive and then apply uniform pressure on the CFRP fabric surface using a roller to help the epoxy adhesive to be absorbed by the CFRP fabric. Attach two aluminium plates to both sides of the CFRP/SMA patch to provide a uniform thickness for the patch;
(i) Place the top flat steel plate over the top layer of the CFRP fabric. Press the top steel plate to remove the excess epoxy adhesive and to maintain a uniform epoxy adhesive thickness between the top CFRP layer and bottom CFRP layer. The patch is left to cure at room temperature for a minimum of seven days.

Figure 2 Fabrication process for the third type (CFRP-(2+1)L/15BT-SMA(NP)), the fourth type (CFRP-(2+1)L/9BT-SMA(P)) and the fifth type (CFRP-(2+1)L/15BT-SMA(P)) of patch

Attaching the CFRP and CFRP/SMA Patches on the Steel Plates

The five types of fabricated CFRP and CFRP/SMA patches were attached to steel plates 500 mm in length, 90 mm in width and 6 mm in thickness. The procedures followed for attaching the CFRP fabrics and CFRP/SMA patches to the initially cracked steel plates are listed below and shown in Figure 3:
(a) Clean the initially cracked steel plate using acetone and then sand-blast it to increase its surface roughness for better bonding between the CFRP fabric patch or the CFRP/SMA patch and the steel plate. Place the sand-blasted steel plate on a flat bottom steel plate and then apply the epoxy adhesive to the sand-blasted area;
(b) Apply the epoxy adhesive to one side of the CFRP fabric patch or CFRP/SMA patch;
(c) Place the epoxy side of the CFRP fabric patch or CFRP/SMA patch on the initially cracked steel plate;

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(d) Place the top flat steel plate over the CFRP fabric patch or CFRP/SMA patch. Press the top steel plate to remove the excess epoxy adhesive and to maintain a uniform epoxy adhesive thickness between the CFRP fabric patch or CFRP/SMA patch and the initially cracked steel plate. The whole set-up is left to cure at room temperature for a minimum of seven days.

![Figure 3 Attachment process of the CFRP fabrics and CFRP/SMA patches to the initially cracked steel plates](image)

**Fatigue Tests on the Un-strengthened and Strengthened Initially Cracked Steel Plates**

Fatigue tests were conducted on the un-strengthened initially cracked steel plates and the initially cracked steel plates strengthened with CFRP fabric and CFRP/SMA patches using a 250 kN MTS testing machine to determine their fatigue life, defined as the number of loading cycles required to fracture the specimens. During the fatigue tests, the specimens were subjected to a cyclic loading range of 9.2 kN - 92 kN for 60000 cycles at a frequency of 10 Hz followed by a cyclic loading range of 9.2 kN - 50.6 kN for 30000 cycles at a frequency of 20 Hz, and this cyclic loading is repeated until the total fracture of the specimen. For the initially cracked steel plates strengthened with CFRP/SMA patches, the pre-strained SMA wires embedded inside the patches were activated using the heat produced by halogen lamps to generate stress recovery before applying the cyclic loading. The initially cracked steel plates were prepared following the procedure given by the ASTM Standard E647 (2013). The notch in the steel plate was located in the centre of the steel plate for the purpose of specifying the place where the steel plate should fracture. The experimental test set-up for fatigue testing using the MTS testing machine and a schematic of the initially cracked steel plates are shown Figure 4.

![Figure 4 Experimental test set-up for fatigue testing using the MTS testing machine and a schematic of the initially cracked steel plates](image)
RESULTS AND DISCUSSION

From the fatigue tests conducted on the un-strengthened initially cracked steel plates and the initially cracked steel plates strengthened with CFRP fabrics and CFRP/SMA patches, the fatigue life of the specimens was determined. Six groups of specimens with two repeats for each group were prepared and tested under cyclic loading fatigue. The first group was the un-strengthened initially cracked steel plates which was the control group and given the name SP-(Control). The second group was the initially cracked steel plates strengthened on both sides using the first type (CFRP-2L) of patch and was given the name SP-(CFRP_2L2S). The third group was the initially cracked steel plates strengthened on both sides using the second type (CFRP-(2+1)L) of patch and was given the name SP-(CFRP_(2+1)L2S). The fourth group was the initially cracked steel plates strengthened on both sides using the third type (CFRP-(2+1)L/15BT-SMA(NP)) of patch and was given the name SP-(CFRP_(2+1)L2S)-(15BT_SMA(NP)). The fifth group was the initially cracked steel plates strengthened on both sides using the fourth type (CFRP-(2+1)L/9BT-SMA(P)) of patch and was given the name SP-(CFRP_(2+1)L2S)-(9BT_SMA(P)). The sixth group was the initially cracked steel plates strengthened on both sides using the fifth type (CFRP-(2+1)L/15BT-SMA(P)) of patches and was given the name SP-(CFRP_(2+1)L2S)-(15BT_SMA(P)). The fatigue life test results for the six groups of the un-strengthened and strengthened initially cracked steel plate specimens are shown in Figure 5.

From Figure 5, by comparing the fatigue life of the first and second group, it can be seen that the fatigue life of the initially cracked steel plate is significantly increased by an increment factor of 2.97, due to the effect of the attached CFRP fabric patches. By comparing the fatigue life of the second, third and fourth groups, it can be seen that the fatigue life of the initially cracked steel plate is slightly increased, which indicates that adding the 90° ply CFRP fabric to the composite has a very small effect on increasing the fatigue life of the initially cracked steel plate. The reason for adding the 90° ply CFRP fabric to the CFRP/SMA composite was to prevent the SMA wires from peeling out of the CFRP/SMA patch during the activation process required to generate the recovery stress using the halogen lamps. It also indicates that embedding the non-pre-strained body temperature NiTi SMA wires inside the CFRP/SMA patches has no effect on increasing the fatigue life of the initially cracked steel plate. On the other hand, by comparing the fatigue life of the fifth and sixth groups with that of the fourth group, it can be seen that embedding the pre-strained body temperature NiTi SMA wires inside the CFRP/SMA patches can effectively increase the fatigue life of the initially cracked steel plates.

Figure 5 Fatigue life results of the un-strengthened and strengthened initially cracked steel plate specimens
CONCLUSION

This paper presents the development of an active hybrid CFRP/SMA composite made of unidirectional normal modulus carbon fibre reinforced polymer (CFRP) fabric embedded with pre-strained body temperature nickel titanium (NiTi) shape-memory alloy (SMA) wires. The study mainly involved the fabrication of CFRP fabrics and CFRP/SMA patches. The patches were then attached using a strong structural epoxy adhesive to initially cracked steel plates. The experimental work included conducting cyclic fatigue testing on un-strengthened initially cracked steel plates, cyclic fatigue testing on initially cracked steel plates strengthened with CFRP fabric patches, and cyclic fatigue testing on initially cracked steel plates strengthened with CFRP/SMA patches. From the results of the fatigue life tests conducted on the un-strengthened and strengthened initially cracked steel plate specimens, it was found that the fatigue life of the initially cracked steel plates was increased by an increment factor of 2.97, 3.19 and 3.17 for the second, third and fourth groups of specimens, respectively. In addition, and most importantly, it was found that the fatigue life of the initially cracked steel plates was increased by an increment factor of 4.0 and 4.76 for the fifth and sixth groups of specimens, respectively. This strongly indicate that embedding pre-strestrained body temperature NiTi SMA wires inside the CFRP/SMA patches used to strengthen initially cracked steel plates can effectively increase their fatigue life.

REFERENCES

PRESTRESSING SYSTEMS FOR STRENGTHENING OF CONCRETE AND METALLIC STRUCTURES: RECENT DEVELOPMENTS AT EMPA, SWITZERLAND

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ABSTRACT

Carbon fiber reinforced polymer (CFRP) materials have been used for strengthening of concrete and metallic structures. There are several differences between the behavior of bonded joints in CFRP-strengthened concrete and metallic members. One of the main differences between CFRP–concrete and CFRP–metal bonded joints is that in the latter, failure will likely occur in the adhesive layer and in the former failure is expected to occur in the concrete. Furthermore, in the concrete girders, cracks are often initiated at low load levels in the tension face, and the bonded CFRP strip tries to close the crack, and consequently increase the stiffness and cracking load. Nevertheless, metallic girders do not crack even after yielding and the effect of having adhesive between the CFRP laminate and steel substrate is limited to transferring the shear stresses from the steel substrate to the CFRP laminate along the connection. Because of these differences, the strengthening concepts for steel and concrete are different, and, therefore, different CFRP prestressing systems are required. This paper provides a short review for different CFRP retrofit systems that have been recently developed at Empa for concrete and metallic members. Details about strengthening of a concrete and a metallic bridge using prestressed CFRP strips are given. At the end, a novel pre-stressing system, which is based on iron-based shape memory alloy (Fe-SMA), is presented.

KEYWORDS

Carbon-fiber-reinforced polymer (CFRP), prestressing, steel structures, reinforced concrete, strengthening, iron-based shape memory alloy (Fe-SMA).

INTRODUCTION

History of CFRP Composites for Structural Strengthening

Application of adhesively bonded fibre composite reinforcement for strengthening of aircraft structures has begun in mid-1870s (i.e., Baker 1975). Carbon-fiber-reinforced polymer (CFRP) or boron-fiber-reinforced polymer (BFRP) were used for strengthening of fatigue- and corrosion-cracked aircraft structures made of aluminum alloys (e.g., Baker 1978).

About twenty years after the first CFRP-strengthening application in aerospace engineering, the application of CFRP materials for rehabilitation of civil structures has been started by Professor Urs Meier and his team in mid-1890s, for the first time, at the Swiss Federal Laboratories for Materials Science and Technology (Empa). Meier and his (e.g., Meier 1987, Meier 1995, Meier and Deuring 1991) team have done the first studies worldwide on the use of CFRP composites for strengthening of civil structures.

Nowadays, the use of CFRP composites is common for strengthening of concrete (e.g., Czaderski and Motavalli 2007, Michels et al. 2014, Motavalli et al. 2011) and metallic (e.g., Ghafoori and Motavalli 2015b, Ghafoori and Motavalli 2015c, Ghafoori and Motavalli 2015a) structures. Despite of the common use of CFRP materials for civil strengthening applications, only 20 to 30% of the high strength of CFRP strips is used (Motavalli et al. 2011). One reason that can be contributed to this poor material efficiency is that most strengthening tasks are stiffness controlled rather than strength-controlled (Meier 2007). Therefore, a more efficient use of the high strength of the CFRP is achieved by prestressing the CFRP material.

Prestressed CFRP Strips for Structural Strengthening

In the early 1990s, the first worldwide research on prestressing CFRP strips for strengthening of reinforced concrete structures was initiated by Meier and his team (Meier 1995). In this research, CFRP strips were prestressed using different prestressors such as conventional concrete prestressing tendons, cold-formed stainless steel wires, and glass and carbon fiber composite tendons. The investigation showed that prestressing of CFRP strips could increase the strain capacity of the strips and lead to a better utilization of the high strength of the CFRP material. However, the use of conventional prestressing tendons is limited by the high initial cost and the need for dedicated prestressing equipment. Furthermore, the use of cold-formed stainless steel wires is limited by the low strength of the wires and their poor durability in adverse environments.

In order to overcome these limitations, new prestressing systems have been developed and reported in the literature. These systems include the use of self-locking steel bolts, mechanical devices, and shape memory alloy (SMA) wires. Among these systems, the use of SMA wires has attracted significant interest due to their ability to withstand high temperatures and to provide a high level of resistance to corrosion.

The use of SMA wires for prestressing CFRP strips has several advantages. First, SMA wires can be easily manufactured with a high level of resistance to corrosion and high temperature. Second, SMA wires can be easily installed using conventional prestressing equipment. Third, SMA wires can be easily adjusted to the desired prestress level, which is an important advantage when prestressing CFRP strips for strengthening of concrete structures.

In this paper, a novel pre-stressing system, which is based on iron-based shape memory alloy (Fe-SMA), is presented. This system allows for the easy installation of the prestressors and provides a high level of resistance to corrosion. The prestressors can be easily adjusted to the desired prestress level, which is an important advantage when prestressing CFRP strips for strengthening of concrete structures.

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In summary, the use of Fe-SMA wires for prestressing CFRP strips has several advantages over other prestressing systems. First, Fe-SMA wires can withstand high temperatures and provide a high level of resistance to corrosion. Second, Fe-SMA wires can be easily installed using conventional prestressing equipment. Third, Fe-SMA wires can be easily adjusted to the desired prestress level, which is an important advantage when prestressing CFRP strips for strengthening of concrete structures.
Concrete (RC) girders was started at Empa (Deuring 1993). In this study, mechanical anchorages were used to attach the prestressed CFRP strips to the RC girders. After some years, also at Empa, the gradient end anchorage technique was invented and patented by Meier and his team (e.g., Meier and Stocklin 2005, Stocklin and Meier 2001, Stocklin and Meier 2002). More recent developments in the field are given in Ghafoori and Motavalli 2016b. The current paper aims to present the different prestressing techniques that have been developed recently at Empa for strengthening of concrete and also metallic structures. The strengthening material for the majority of the applications was the CFRP composites, however, this paper also presents a new prestressing system, which is based on using iron-based shape memory alloy (Fe-SMA). The paper has three main sections. Each section focuses on a specific strengthening topic. The first section gives a brief review on the Empa research on strengthening of RC girders. The second section focuses on CFRP strengthening of metallic members, and the third section gives a short overview about the ongoing research at Empa on development and application of Fe-SMA products as a new prestressing member in civil engineering.

**STRENGTHENING OF RC GIRDERS WITH PRESTRESSED CFRP STRIPS**

When RC girders are strengthened with prestressed bonded CFRP strips, prestressing decreases existing crack widths, deflections and stresses, and hence enhances durability and serviceability of the retrofitted member. Moreover, prestressed bonded reinforcement (PBR) system decreases the stress range in the steel rebars, and, therefore, it improves the fatigue behavior.

An innovative CFRP prestressing technique without using a mechanical anchorage system for strengthening of RC girders has been developed at Empa. In this method, the prestressing force in the CFRP plate is gradually decreased to zero at both end of the strip, with intermediate sector-wise accelerated curing of the epoxy resin. This method uses a special prestressing and heating device so that there is no need for a mechanical anchorage system. As mentioned, the gradient in the prestress force in the CFRP strip is applied by a sector-wise heating and curing of the adhesive, while the force in the CFRP strip is released step-wise, as illustrated in Figure 1. The efficiency of such a retrofitting system with gradient anchorage system was demonstrated by static loading tests on 17-m-
long concrete girders taken from an existing bridge in Ticino (Switzerland) (Czaderski and Motavalli 2007). The maximum loads reached with the girders were 24% (for non-prestressed CFRP, girder 4) and 45% (for prestressed CFRP, girder 2) higher than that of the reference girder (girder 3), as shown in Figure 2.

Tulcoempa project: Similar experimental investigation has been performed in the framework of a Polish-Swiss Research project ‘Tulcoempa’. Prior to strengthening of existing prestressed concrete girders on an existing bridge in Poland, two girders were re-produced at Empa according to the original drawings. Subsequently, one girder was retrofitted with two prestressed CFRP strips (total prestress force 2 x 120 kN=240 kN) with a gradient anchorage. A shear strengthening with a CFRP fabric was also applied. As shown in Figure 4, an increase in cracking, yielding and ultimate load could be reached. Additionally, the ultimate load was reached by tensile failure in the CFRP strip at around 1.6% strain Gafoori and Motavalli 2016a.

Similar girders of a bridge in Poland were afterwards strengthened, also with prestressed CFRP laminates with gradient anchorage. The sole difference in this case was a reduced prestress force per strip of 75 kN and 90 kN, respectively. The existing upper concrete deck was also replaced in the framework of this reparation work. A photo of the bridge is given in Figure 5.

**Figure 5.** a) The road RC bridge in Poland (b) bridge RC girders after strengthening.

**STRENGTHENING OF METALLIC STRUCTURES WITH CFRP STRIPS**

As it has been discussed in the last section, application of CFRP materials for retrofitting concrete girders has been extensively investigated and used in practice. Many studies demonstrated the beneficial influence of such composite strips for flexural, shear and confinement strengthening of concrete structures. However, strengthening techniques and the accompanying theory for metallic structures have not been developed as thoroughly as those for concrete structures. There are several differences between the behavior of bonded joints in CFRP-strengthened concrete and metallic members, which will be briefly explained in this section.

**Differences between Strengthening of Concrete and Steel Members**

*Failure mode:* The main difference between CFRP–steel and CFRP–concrete bonded joints is that in the former, failure will likely occur in the adhesive layer and in the latter failure is expected to occur in the concrete. Therefore, by providing an adequate bond length, the optimal strength of a bond joint is dependent on the fracture energy of the adhesive for the former and the fracture energy of the concrete for the latter. Based on Teng 2012, in the FRP-strengthened steel structures, interfacial failure should happen within the adhesive layer in the form of cohesion failure to maximize the effectiveness of FRP strengthening and minimize variations of the interfacial bond capacity as a result of different surface preparations. Furthermore, Teng 2012 reported that the inappropriate surface preparation of the steel substrate prior to the bond application will result in an adhesion failure at the steel-to-adhesive interface. Assuming the adhesive as the weakest point of a CFRP-steel bond joint, Gafoori et al. [15,16] have developed a prestressed unbonded reinforcement (PUR) system that can be used as an alternative to the bonded CFRP reinforcement.

*Stiffness and deformations:* Recent experimental and numerical results studies at Empa (Gafoori and Motavalli 2015b, Gafoori and Motavalli 2015c) have shown that pre-stressing the CFRP strips does not increase the stiffness of the retrofitted steel beams. This finding is in contrast to retrofitted concrete beams, in which the cracks are often initiated at even low load levels. In the latter case, pre-stressed laminates can close the existing cracks more efficiently than non-pre-stressed laminates, and thereby, the overall cross-section area of the concrete beam is increased, which results in increased stiffness. For the same reason, pre-stressed laminate can increase the stiffness of the cracked steel members.
Strengthening Steel Members using Prestressed Bonded and Un-bonded CFRP Strips

Bonded Systems: Ghafoori and Motavalli 2015c have used bonded non-pre-stressed normal modulus (NM), high modulus (HM) and ultra-high modulus (UHM) for flexural strengthening of steel beams. It has been shown that UHM CFRP strip are effective in increasing the stiffness of the metallic girders and reducing the deformations. Metallic members have been traditionally strengthened using non-pre-stressed CFRP plates. However, in non-pre-stressed retrofit systems, the dead loads are not transferred to the CFRP plates and only a portion of the live load is transferred to the CFRP plates. As an alternative, by using pre-stressed CFRP plates, a portion of the dead load is transferred to the CFRP plates in addition to the live load (Ghafoori 2013, Ghafoori and Motavalli 2013). It has been shown that prestressed CFRP strips can increase the flexural yield and ultimate load capacity of steel beams substantially. Ghafoori and Motavalli 2015b have shown that prestressed CFRP strips can be used for strengthening of steel beams that are prone to lateral-torsional buckling (LTB) to increase the LTB strength. Moreover, Ghafoori et al. 2012b studied the performance of notched steel beams retrofitted with CFRP patches under high-cycle fatigue loading regime. The test results for a four-point bending test scheme with a cyclic loading frequency of 4.2 Hz showed that the CFRP patch extended the fatigue life substantially, and in some case, a complete fatigue crack arrest was achieved.

Un-bonded Systems: The majority of the existing research on CFRP strengthening of metallic members has used CFRP material bonded to the steel substrate. As it has been discussed before, the efficiency of the bonded retrofit system is mainly dependent on the behavior of the CFRP-to-steel bond joint. Sophisticated surface preparation is required prior to bonding the CFRP to the steel member to maximize the efficiency of the composite system and reduce the risk of debonding. Many studies have raised concerns about the influence of environmental conditions (e.g., elevated or subzero temperatures, water and moisture and ultraviolet light) and dynamic loads (e.g., fatigue, impacts and earthquakes) on the behavior of the CFRP-to-steel bond joint (e.g., Al-Zubaidy et al. 2012, Dawood and Rizkalla 2010, Wu et al. 2013). Because of these concerns, which are mainly associated with the long-term performance of the CFRP-to-steel bond joints, a pre-stressed un-bonded retrofit (PUR) system has been recently designed and tested at Empa (Ghafoori and Motavalli 2015b, Ghafoori and Motavalli 2015c, Ghafoori and Motavalli 2015a). In contrast to the PBR system, the PUR system works without using any bond; instead, it uses a pair of friction clamps to connect the CFRP plates to the steel member. An independent reaction frame to pull the CFRP strips was developed, as shown in Figure 6. The pre-stressed CFRP strip was then attached to the steel beam using mechanical clamps. The force in the actuator was then released and the CFRP strip out of mechanical clamps was cut.

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Figure 6. Elements of the pre-stressing set-up, which uses an independent reaction frame to pull the CFRP strip (Ghafoori and Motavalli 2015b).

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The retrofitted beams were tested in a four-point bending static loading test set-up, as shown in Figure 7.a. It has been shown that prestressed unbonded and bonded CFRP strip have almost identical effect on the behavior of steel beams. Prestressed unbonded CFRP strips could prevent fatigue crack initiation (Ghafoori et al. 2015b) and propagation (Aljabar et al. 2016, Ghafoori et al. 2012a) in steel beams. In summary, the results of the extensive tests have shown that the static and fatigue behavior of steel beams are strongly governed by the prestress level in the CFRP strip, rather than the effect of the adhesive bond (Ghafoori and Motavalli 2016b). Bonded and unbonded systems have shown relatively similar results, particularly in the elastic domain (Ghafoori 2015).

**Trapezoidal PUR system:** Figure 7.a shows a PUR system with straight CFRP strips. Ghafoori and Motavalli 2015a have recently developed a trapezoidal PUR system for strengthening of a historic metallic railway bridge in Switzerland. A summary of the prestressing procedure is explained as follows. Assume an I-beam as shown in Figure 7.b (i). First, the mechanical clamps are placed near two ends of the beam, and three parallel CFRP plates are placed and tightened inside the clamps, as shown in Figure 7.b (ii). Each CFRP plate has dimensions of 50 mm width and 1.2 mm thickness. Each friction clamp is consisted of a lower plate, a middle plate and two upper plates. The middle and the lower plates consist of three hard plates, which provide a uniform stress distribution along the CFRP anchorage length. Each CFRP plate is anchored between the lower plate and the middle plate and is subjected to compressive force, which is applied by pre-tensioned bolts. The beam flange is also gripped between the middle plate and the upper plates and subjected to the compressive force of pre-tensioned bolts. A pre-stressing chair is used to increase the eccentricity between CFRP plates and steel beam, as shown in Figure 7.b (iii).

![Figure 7](image-url)  
(a) Trapezoidal PUR system with straight CFRP strips: (a) Example of a PUR system with straight CFRP strips. (b) The cross-girders were retrofitted with pre-stressed unbonded CFRP strips. (c) The prestressing chair consists of a saddle that can move along two vertical threaded bars. The distance between the saddle and the beam can be manually changed by turning the threaded rods using a wrench. Thus, by turning the threaded rods, the saddle pushes the CFRP plates away from the beam, and the CFRP pre-stress is increased. A larger eccentricity between the CFRP plates and the beam corresponds to a larger CFRP pre-stress level. After the desired pre-stress level is achieved, two plates are placed between the CFRP plates and the beam (see Figure 7.b (iv)). Each plate is positioned between the saddle and a shoe. The two shoes are connected by two steel bars and four nuts, as shown in Figure 7.b (v), and then the pre-stressing chair is removed. Figure 7.b (v) shows the final configuration of the strengthened beam. More details can be found in Ghafoori and Motavalli 2015a, Ghafoori et al. 2015a.

After the trapezoidal PUR system has been tested under static and fatigue tests in laboratory, the system has been used for fatigue strengthening of a 120-year-old railway metallic bridge in Switzerland, as shown in Figure 8.a and 8.b (Ghafoori et al. 2014). In order to monitor the performance of the girder and retrofit system, a wireless sensor network (WSN) system was used for about one year, which showed the successful implementation of the system. The PUR systems can have different other configurations, as has been explained by Kianmofrad et al. 2016.

**STRENGTHNEING USING IRON-BASED SHAPE MEMORY ALLOYS**

Shape memory alloys (SMAs) have several unique properties. The most important properties are the shape memory effect and the superelasticity (Janke et al. 2005). Iron based shape memory alloys were discovered by Sato et al. Sato et al. 1982. For civil engineering applications, Fe-based shape memory alloys represent a promising technology because of their properties and lower cost when compared to NiTi. An iron-based shape memory alloy

![Figure 8](image-url)
(Fe–17Mn–5Si–10Cr–4Ni–1(V,C) (ma.–%)) has been developed and patented at Empa (Dong et al. 2009). In fact, in the early 2000s at Empa, possible application ideas were mainly for NiTi-SMAs such as controlled or fixed prestressing, superelasticity with high energy dissipation or for sensors (Czaderski and Motavalli 2003). In 2003, a concrete beam was reinforced with shape memory alloys (SMA) wires (Czaderski et al. 2006). NiTi wires were used to reinforce the tensile zone of a concrete beam with a span of 1.14 m. In 2003-2004, shape memory alloy (SMA) NiTi wires were embedded in mortar to demonstrate the feasibility of prestressed short fiber reinforced concrete (Moser et al. 2005).

The new Fe-based SMA for civil engineering applications has been developed at Empa during 2005-2009 (Dong et al. 2009) and patented in 2009. A feasibility study on the usage of the developed Fe-SMA for the strengthening of reinforced concrete structures has been done in 2013 (Czaderski et al. 2014). Furthermore, lap-shear tests on the Fe-SMA strip that were glued in grooves using cement based grout were performed (Czaderski et al. 2014), which resulted in a good bond behavior.

![Figure 9](image1.png)

**Figure 9.** Setup for activation of strengthened beams by Fe-SMA strips/bars (Shahverdi et al. 2016b): (a) Fe-SMA strips in grooves, (b) Fe-SMA bars in shotcrete layer.

In another work at Empa Shahverdi et al. 2015, several reinforced concrete (RC) beams were strengthened using the near surface mounted (NSM) reinforcement technique with ribbed Fe-SMA strips (see Figure 9.a). Embedding ribbed Fe-SMA bars in a new shotcrete layer has been done as another promising method to strengthen RC structures Shahverdi et al. 2016a (see Figure 9.b). Similar to NSM, shotcreting is a well-known technique to strengthen and repair existing RC structures. It is therefore an adequate material in combination with the ribbed Fe-SMA bars. In comparison with conventional shotcreting techniques, application of Fe-SMA bars allowed the prestressing of the shotcrete layer. The load-displacement diagrams for some of Beam 1 (reference), NSM strengthened beams (Beams 2, 3 and 6) and prestressed shotcrete beams (Beams 9 and 10) are shown in Figure 10.

![Figure 10](image2.png)

**Figure 10.** Load mid-span displacement diagrams of some the examined beams at Empa.

CONCLUSIONS

The key differences between CFRP strengthening of metallic and concrete structures were discussed. The two main differences between the behavior of CFRP–concrete and CFRP–metal bonded members are concerned with
the failure mode and the stiffness of the retrofitted members. These differences resulted in development of different CFRP prestressing concepts for strengthening of concrete and metallic members at Empa, which were briefly explained in this paper. The application of gradient anchorage system for strengthening of concrete girders with prestressed CFRP strips was explained. The results of laboratory tests as well as details about the filed application on a RC bridge in Poland were briefly discussed.

For strengthening of metallic members, laboratory test results showed that adhesive bond does not have much influence on the static and fatigue behavior of retrofitted steel beams, however CFRP prestressing plays an important role. Therefore, in order to minimize the concerns related to effects of high ambient temperatures, moisture, water or fatigue loading on the CFRP-to-metal bond behavior, a prestressed unbonded retrofit system has been recently developed at Empa. Mechanical anchorage system was used to attach prestressed CFRP strips to metallic members. Details about fatigue strengthening of a 120-year-old railway metallic bridge in Switzerland using prestressed unbonded CFRP strips were given. Finally, ongoing studies at Empa on characterization and application of FE-SMA, a new retrofit material, were briefly explained.

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ABSTRACT

CFRP strengthening of steel structures has proved to be an effective method of repair of metallic structures. Under life time cyclic events cracks can initiate and propagate in metallic structures. In order to repair a fatigue induced crack, an effective repair configuration must be chosen. To meet this requirement, understanding the effect of different strengthening parameters on the crack propagation is required.

CFRP strengthening has been used in this research to retard the crack growth in initially cracked steel plates. 6mm thick steel plates with two different initial notch lengths were chosen in this study.

In order to add to the efficiency of the results, prestressing has been introduced to the CFRP patches. The prestressing load is applied by a hydraulic actuator prior to bonding the CFRP patch to the cracked steel plates. After preparation, the specimens were all tested under a constant amplitude fatigue load.

Fatigue life and the crack propagation measurements is the main outcome of this research. This is while strain gauges were also attached to the specimens and the effect of CFRP type and prestress level are also measured. The results show that the CFRP thickness and modulus of elasticity are effective parameters on the enhancement of the fatigue life. Though, application of prestressed CFRP can significantly improve the fatigue life of the cracked members.

KEYWORDS
Prestressing, CFRP, Crack propagation, Fatigue, Strengthening, Steel.

INTRODUCTION

Overall, there are three main phases where the upgrading or repair process occurs (Hollaway (2014)):

- In order to upgrade the structure as a reason of change in application thus difference in loading conditions of the structure.
- Before any visible damage to prolong the life of the structure.
- After visible damage has occurred in order to repair the structure.

Fibre reinforced polymer composites are now frequently used in civil and structural applications. Utilising the conventional methods of rehabilitation in steel structures had the disadvantage of making the structures heavier by attaching bulky and heavy steel plates. The attached plates are also prone to fatigue and corrosion themselves (Zhao et al. (2007)). To attach these bulky and heavy steel plates a lifting system is required which has to be powerful enough, while the CFRP patches are very light and can be lifted much more easily. Attaching steel plates to the existing structure can be done by welding or bolting. However, the weld is subject to fatigue and using bolts elevates the cost and workmanship required.

Figure 27 shows some examples of cracks that can be repaired using bonded CFRP patches. This figure shows cracks that have initiated from locations with high local stresses and their propagation under imposed traffic loads on the bridge.

Use of bonded CFRP patches has been tested in the literature frequently. Works of Jones et al. (2003), Liu et al. (2009), Wu et al. (2012) have all concluded the effectiveness of CFRP patching on cracked steel plates. However,
some researchers have tested the effect of CFRP patching beyond its passive effect by applying prestressing to the CFRP patches before bonding them to the cracked region.

Täljsten et al. (2009), carried out experimental and numerical investigations on the effect of prestress and non-prestressed CFRP laminates. They tested old steel plates which were taken from an old bridge. In order to simulate the crack propagation they drilled a hole and two notches at the edge of the hole. The notches represent crack initiation. They applied a tension force to the CFRP before bonding them to the steel.

In this paper the experimental work performed includes the study of the passive effect and active effect of CFRP patching on cracked steel plates.

Figure 27 Cracks in a bridge structural member

METHODOLOGY

The fatigue life of initially cracked steel plates is studied in this research. Steel plates with a hole in the centre and two notches emanating from the edges of the hole is a typical sample that has been studied. The notch represents the crack initiation. The centre-notched steel plates are then repaired with CFRP patches. In order to bond the CFRP to the steel, Araldite 420 (a two part adhesive) is used. After the CFRP strengthened steel plates are prepared they are all tested under cyclic fatigue. In some specimens prestressing was applied to the CFRP. Different specimens are prepared and tested as listed below:

- 6 mm CNS plates without repair (Reference Plate)
- 6mm CNS plates repaired with normal modulus CFRP sheets
- 6mm CNS plates repaired with prestressed normal modulus CFRP sheets

The main aim of this research was to understand the behaviour of cracked steel plates under fatigue and identifying suitable repair mechanisms. Therefore, different parameters were studied in the experiments such as:

- Effect of number of layers of CFRP
- Effect of prestressing
- Effect of initial notch length

EXPERIMENTS

Material

In this study 6mm thick steel plates were used. Since the major aim of this study was to measure the cracked life of the specimens, centre-notched steel (CNS) plates were utilized. This means that the load cycles applied to the specimens will contribute to crack growth rather than crack initiation. The dimensions of the CNS plates are given in Figure 28.
Normal modulus CFRP sheets are used in this study. The material properties of the CFRP are given in Table 1. For the means of bonding the CFRP patch to the steel plates Araldite 420 was used. This adhesive has a pot-life of about 1 hour which is a benefit and makes working with the adhesive easier. The mechanical properties of Araldite 420 has a tensile strength of 28.6MPa, shear strength of 36MPa, ultimate strain of 0.024, Elastic modulus of 1901MPa and poison’s ratio of 0.36 (Fawzia et al. (2006)). Araldite 420 curing time in room temperature (25°C) is 7 days. According to the manufacturer’s datasheet, araldite 420 will cure in 1 hour in 100°C.

**Preparation of specimens**

*Set A) Reference Plates*

Details of this set of specimens are shown in Figure 28. A hole with 2 notches emanating from the sides is located at the centre of the plate. The dimensions of the plate are 90mm width and 500mm length. The plates tested in this research were all 6mm thick, with an elastic modulus of 212000 MPa and yield strength of 335 MPa.

![Figure 28 Centre-notched steel (CNS) plate](image)

**Table 1 Mechanical properties of the normal modulus CFRP provided by manufacturer**

<table>
<thead>
<tr>
<th></th>
<th>MBrace CF 230/4900</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fibre reinforcement</td>
<td>Carbon-high tensile</td>
</tr>
<tr>
<td>Fibre density (minimum)</td>
<td>1.76 g/cm³</td>
</tr>
<tr>
<td>Tensile modulus</td>
<td>230 GPa</td>
</tr>
<tr>
<td>MBrace fibre thickness</td>
<td>0.17</td>
</tr>
<tr>
<td>Ultimate tensile strength</td>
<td>4900 MPa</td>
</tr>
</tbody>
</table>

*Initial crack length*

As mentioned earlier, in this practical research the crack growth phase of the fatigue life was of interest. Therefore, the specimens were all initially notched. The notch was emanated from a centre hole in the steel plate. The geometry of the notch is shown in Figure 28. Two different notch lengths were studied in this research for the unstrengthened plates. In one set of specimens the total initial crack length size was 7mm (including the hole diameter) and in the second set of specimens the total crack length was 11.5mm.

*Set B) 6mm CNS plates repaired with normal modulus CFRP sheets*

The strengthening configuration of the specimens is shown in Figure 29. One, two and three layer CFRP strengthening configuration was investigated. The patch dimension was 50mm x 250mm and it was bonded to the steel at the location of the crack.

*Set C) 6mm CNS plates repaired with prestressed normal modulus CFRP sheets*

CFRP strengthening benefitting from prestress was performed in this set of specimens. The prestress levels and the prepared specimens are shown in Table 2.
The strengthening patterns (CFRP location and size) in all the specimens were the same. The only difference was the number of layers and the prestress level applied.

Table 2 Specimen list with prestress levels

<table>
<thead>
<tr>
<th>Description</th>
<th>Label</th>
<th>Prestress*</th>
<th>Stress level on CFRP (MPa)</th>
<th>Stress level on steel (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference plate (Non-reinforced)</td>
<td>BP</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Double-sided repair, 1 layer of CFRP*</td>
<td>D1L</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Double-sided repair, 2 layers of CFRP</td>
<td>D2L</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Double-sided repair, 3 layers of CFRP</td>
<td>D3L</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Double-sided repair, 1 layer of prestressed CFRP</td>
<td>D1LP25</td>
<td>25</td>
<td>1500</td>
<td>46.3</td>
</tr>
<tr>
<td>Double-sided repair, 1 layer of prestressed CFRP</td>
<td>D2LP25</td>
<td>25</td>
<td>750</td>
<td>46.3</td>
</tr>
<tr>
<td>Double-sided repair, 2 layers of prestressed CFRP</td>
<td>D2LP50</td>
<td>50</td>
<td>1500</td>
<td>92.6</td>
</tr>
</tbody>
</table>

#L: Number of layers  P##: Prestress level (kN)  D: Double-sided repair
*Stress levels are calculated based on gross areas.

Table 3 Specimen list considering different notch types

<table>
<thead>
<tr>
<th>#</th>
<th>Description</th>
<th>Prestress Level (kN)</th>
<th>Specimen</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Notch Type (N1)</td>
</tr>
<tr>
<td>1</td>
<td>Reference plate (Non-reinforced)</td>
<td>0</td>
<td>BP(N1)</td>
</tr>
<tr>
<td>2</td>
<td>Double-sided repair, 1 layer of CFRP</td>
<td>0</td>
<td>D1L(N1)</td>
</tr>
<tr>
<td>3</td>
<td>Double-sided repair, 2 layers of CFRP</td>
<td>0</td>
<td>D2L(N1)</td>
</tr>
<tr>
<td>4</td>
<td>Double-sided repair, 3 layers of CFRP</td>
<td>0</td>
<td>D3L(N1)</td>
</tr>
<tr>
<td>5</td>
<td>Double-sided repair, 1 layer of prestressed CFRP</td>
<td>25</td>
<td>D1LP25(N1)</td>
</tr>
</tbody>
</table>

#L: Number of layers  P##: Prestress level (kN)  D: Double-sided repair  N#: Notch type

Specimen preparation

For preparation of the specimens several steps were completed to ensure a quality bond between the CFRP patch and the steel plate. These steps were similar for both types of CFRP used in this research. The first step was to sandblast the steel surface at the area where the CFRP was to be attached. Sandblasting removes rust or paint on the surface and provides a rough surface to provide better interlocking at the bond between the CFRP and steel. The second step was cleaning the surface of the sandblasted steel using acetone. This action helped to remove dust and grease which can harm the bond. When this action was completed the adhesive (in this case Araldite 420) was applied to the desired location. The CFRP was then left for curing. The curing time for all specimens in this study was 7 days at room temperature.

In the specimens for which prestressing was involved, the procedure was quite different, as explained in the next section.
Prestressing

After sandblasting and cleaning the surface of the steel, prestressing of the CFRP was done. The prestressing process had four main stages:
1- Tensioning of the CFRP patch
2- Attaching the patch in tension to the CNS plate
3- Curing of the loaded specimen
4- Unloading of the patch

In this research the pretensioning was done using an MTS hydraulic actuator in force control mode. First the CFRP was located in the grips. The appropriate amount of Araldite was then spread on the CFRP patch with the length being equal to the desired bond length. Then the steel plate was located in place. After this the CFRP patch was pulled up to a desired amount. This action caused tensile levels in the CFRP. While the patch was in tension the desired bond length of the CFRP formed a bond between the CFRP and the steel. It was ensured that the CFRP and steel plate were aligned perfectly in order to prevent eccentric loading. The specimen was then left under load for 5-7 days for curing. After curing, the load was removed from the CFRP, causing compressive stress formation in the steel. The extra CFRP was then cut off.

Fatigue loading

After preparing, the specimens were tested under cyclic fatigue. The stress ratio for the fatigue tests was set 0.1. The specimens were tested under two different loads. The details of the cyclic loads are given in Table 4. The loads were both sinusoidal waves applied to the specimen continuously. As seen in Figure 32, there were two blocks in each load type. The reason for using a double block fatigue load was to benefit from the beach marking technique to measure the crack propagation. The first block had a stress range double that of the second block. The crack growth in the specimen was mainly caused by the first block, while when the second block was applied, striations were created on the crack front, representing the crack growth length to the specific number of cycles that were applied. As the fatigue load was applied the crack propagated from the notch through the width of the plate.

The specimens investigated had two different initial notch lengths: specimens with notch type (N1) and specimens with notch type (N2). To compare the effect of the initial crack length, these specimens were tested under load II.

RESULTS

Fatigue life

All the prepared specimens were tested under uniaxial tensile fatigue under the cyclic loads shown in Figure 32. The fatigue lives of all specimens are tabulated in Table 5 and Table 6. The results of the CFRP strengthening show the effectiveness of the patch repair technique on the cracked life of the CNS plates. As the number of CFRP layers increases, the fatigue life increases. This is due to stress reduction at the crack tip.

In the specimens strengthened with prestressed CFRP, the fatigue life enhancement extends up to 35 times. The effect of prestressing is even greater than increasing the number of CFRP layers. Prestressing can contribute to the enhancement of fatigue life in two ways, depending on the level of prestress compared with the fatigue load levels (Huawen et al. 2010). Assuming the crack does not propagate when the stress levels are negative (the specimen is in compression), and if the prestress level is high enough to move the stress range to compression–tension, as is the case for all specimens tested in this study, the stress range reduction contributes to the fatigue life improvement. If the stress range remains in the positive zone (tensile-tensile fatigue), reduction in the stress ratio improves fatigue life. The crack length versus number of cycles is plotted for all the specimens in Figure 30 and Figure 31.

Beach marking technique

To trace the crack propagation as the number of cycles increases, the beach marking technique is used. In this technique a double block fatigue load as explained earlier is applied to the specimens. The first block has a stress range double the stress range of the second block. Therefore, the first block contributes much more to the crack propagation, while the crack propagation due to the second block causes dark marks on the crack front, as shown in Figure 33.
Table 4: Summary of double block fatigue loads applied to all specimens

<table>
<thead>
<tr>
<th>Load type</th>
<th>Specimens</th>
<th>BLOCK 1</th>
<th>BLOCK 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>6mm CNS Plate with</td>
<td>8.1</td>
<td>81</td>
</tr>
<tr>
<td>II</td>
<td>6mm CNS Plate with</td>
<td>9.2</td>
<td>92</td>
</tr>
</tbody>
</table>

Table 5: Comparison of fatigue life of specimens tested under load I and load II

<table>
<thead>
<tr>
<th>Specimens tested under load I</th>
<th>Number of loading cycles to failure</th>
<th>Fatigue life extension ratio</th>
<th>Specimens tested under load II</th>
<th>Number of loading cycles to failure</th>
<th>Fatigue life extension ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>BP-I</td>
<td>218,613</td>
<td>1</td>
<td>BP-II</td>
<td>133,368</td>
<td>1</td>
</tr>
<tr>
<td>D1L-I</td>
<td>632,761</td>
<td>2.89</td>
<td>D1L-II</td>
<td>362,554</td>
<td>2.72</td>
</tr>
<tr>
<td>D2L-I</td>
<td>786,722</td>
<td>3.60</td>
<td>D2L-II</td>
<td>441,187</td>
<td>3.31</td>
</tr>
<tr>
<td>D3L-I</td>
<td>1,029,987</td>
<td>4.71</td>
<td>D3L-II</td>
<td>703,384</td>
<td>5.27</td>
</tr>
<tr>
<td>D1LP25-I</td>
<td>1,574,483</td>
<td>7.20</td>
<td>D1LP25-II</td>
<td>608,857</td>
<td>4.57</td>
</tr>
<tr>
<td>D2LP25-I</td>
<td>1,753,764</td>
<td>8.02</td>
<td>BP-II</td>
<td>786,345</td>
<td>5.90</td>
</tr>
<tr>
<td>D2LP50-I</td>
<td>7,704,456</td>
<td>35.24</td>
<td>D1L-II</td>
<td>2,867,178</td>
<td>21.50</td>
</tr>
</tbody>
</table>
Figure 32: Double-block fatigue load applied to specimens; a) Load I, b) Load II Emdad et al. (2015)

Figure 33: Crack front of some of the tested specimens; a) BP-N1; b) BP-N2; c) D1L-N2; d) D1LP25-N2

Table 6: Comparison of fatigue life of specimens with different notch types tested under load II

<table>
<thead>
<tr>
<th>Specimen with notch type N1</th>
<th>Number of loading cycles to failure</th>
<th>Fatigue life extension ratio</th>
<th>Specimen with notch type N2</th>
<th>Number of loading cycles to failure</th>
<th>Fatigue life extension ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>BP-II (N1)</td>
<td>133,368</td>
<td>1</td>
<td>BP-II (N2)</td>
<td>84,814</td>
<td>1</td>
</tr>
<tr>
<td>D1L-II (N1)</td>
<td>362,554</td>
<td>2.72</td>
<td>D1L-II (N2)</td>
<td>184,783</td>
<td>2.18</td>
</tr>
<tr>
<td>D2L-II (N1)</td>
<td>441,187</td>
<td>3.31</td>
<td>D2L-II (N2)</td>
<td>266,538</td>
<td>3.14</td>
</tr>
<tr>
<td>D3L-II (N1)</td>
<td>703,384</td>
<td>5.27</td>
<td>D3L-II (N2)</td>
<td>423,100</td>
<td>4.99</td>
</tr>
<tr>
<td>D1LP25-II (N1)</td>
<td>608,857</td>
<td>4.57</td>
<td>D1LP25-II (N2)</td>
<td>420,838</td>
<td>4.96</td>
</tr>
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</table>

*Stress levels are calculated based on gross areas.

CONCLUSIONS

As the results show, the fatigue life of the specimens under load II which acquires a higher stress range is lower than that of specimens tested under load I. In the specimens without prestress the average decrease in the fatigue life of the specimens from when they are tested under load I to when they are tested under load II is about 0.6.
Table 6 shows that the initial notch length affects the fatigue life of the specimens. The longer the initial crack, the shorter the fatigue life. In order to compensate the reduction in the fatigue life due to the effect of a longer crack, either additional layers of CFRP can be used or prestress can be applied.

The results of this research indicate that CFRP strengthening of cracked steel members can be used for crack growth retardation. Furthermore, prestressing can be applied to make the strengthening more effective by using the full capacity of the CFRP patch. The effect of initial crack size investigated in this study suggests that postponing strengthening of early cracks in structural elements can cause a significant decrease in the life of the member. The study showed that using prestressed CFRP patches can compensate the fatigue life loss indicated above.

ACKNOWLEDGEMENTS

The authors thank the technical staff of the Smart Structures Laboratory and the workshop at Swinburne University of Technology.

REFERENCES


NUMERICAL STUDY ON FATIGUE BEHAVIOR OF STEEL PANELS WITH INCLINED CRACK REPAIRED BY PRESTRESSED FRP

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ABSTRACT

In this paper, mixed-mode fatigue crack propagation of steel panel was analysed by boundary element method. Both maximum circumferential stress criterion and minimum stress energy density criterion were used to predict crack propagation. Thereafter, FRP patched specimens were modelled and analysed. The effect of FRP modulus was investigated. Prestressing technique was introduced to improve repairing efficiency. Several prestressing levels were considered. Numerical results showed significant crack retardation was achieved with the increase of prestress.

KEYWORDS

Steel panel, fatigue, FRP, prestress, inclined crack.

INTRODUCTION

Fatigue fracture usually occurs in steel structures which are subjected to repeated loads. It happens suddenly without macroscopic plastic deformation. Therefore, effective methods are necessary to repair fatigue cracks. Fiber reinforced polymer (FRP) is widely used in strengthening of steel structures for the superiority of adhesively bonded composite repair method. Liu et al. and Yu et al. performed a series of numerical and experimental researches to study the fatigue behavior of CFRP repaired steel panels with initial notch (Liu et al. 2009; Yu et al. 2013, 2014, 2016). Results revealed that patching CFRP plates can improve the fatigue performance of steel panels. And crack growth can be simulated accurately by boundary element method (BEM) software BEASY. Emdad et al. investigated the effect of prestressed CFRP patched on crack growth of center-notched steel plates (Emdad et al. 2015). They found that prestressing can delay the fatigue crack propagation significantly. However, the previous research only focused on the growth of pure mode I fatigue crack. In practical steel structures, fatigue crack is usually mixed-mode I-II crack. Therefore, Alegre et al. simulated the mixed-mode fatigue crack propagation using Tanaka formula and maximum circumferential stress criterion (MCSC) by finite element method (FEM) (Alegre et al. 2010). The numerical and experimental results consist with each other. However, It’s tedious to remodel and remesh step by step during crack growth simulation by FEM software. This paper examined fatigue behavior of prestressed and non-prestressed FRP repaired steel panels with initial inclined crack by BEM. MCSC and minimum stress energy density criterion(MSEDC) were compared. Results revealed that crack growth criteria have negligible influence on the crack shape when mode I crack plays a dominant role. It was also observed that fatigue life was extended significantly with FRP patching. When prestressed FRP was used to repair the fatigue crack, pretension should be large enough to keep the crack closed under the minimum load, otherwise it has no effect.

NUMERICAL WORK

Crack Propagation Rate and Material Properties

BEASY, a commercial BEM software, was employed in this study to analyze crack propagation. The crack propagation life was integrated using Paris Law as expressed in Eq.(1), where $\Delta K_{eff}$ is effective mixed-mode stress intensity factor (SIF) and calculated by Tanaka Formula as expressed in Eq.(2). Material properties of steel are revealed in Table 1 (BSI 2005). $\Delta K_{II}$, in Eq.(2), should be the positive part of entire $\Delta K_{I}$. Crack may keep closed under minimum load when steel panels are repaired by prestressed FRP. In this case, $K_I$ is negative and should not be accounted. Considering a wide variety of FRP can be applied to strengthen steel structure (YU et al. 2014; MEI et al. 2001; WANG et al. 2015), two kinds of FRP were used in this paper, as shown in Table 2.
\[
\frac{da}{dN} = C(\Delta K_{\text{eff}})^m \tag{1}
\]

\[
\Delta K_{\text{eff}} = \sqrt{(\Delta K_I)^4 + 8(\Delta K_{II})^4}
\tag{2}
\]

\[
\Delta K_I = K_{\text{I max}} - \max(K_{\text{I min}}, 0)
\tag{3}
\]

Table 7 Material properties of steel (N, mm)

<table>
<thead>
<tr>
<th>Material</th>
<th>Young’s Modulus</th>
<th>Poisson’s Ratio</th>
<th>C</th>
<th>m</th>
<th>ΔK_{th}</th>
<th>K_{c}</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>208000</td>
<td>0.3</td>
<td>6.77 x 10^{-13}</td>
<td>2.88</td>
<td>148.1</td>
<td>4170</td>
</tr>
</tbody>
</table>

Table 8 Material properties and dimensions of FRP (N, mm)

<table>
<thead>
<tr>
<th>Material</th>
<th>Young’s Modulus</th>
<th>Poisson’s Ratio</th>
<th>Tensile Strength</th>
<th>Thickness</th>
<th>Width</th>
</tr>
</thead>
<tbody>
<tr>
<td>FRP-1</td>
<td>95500</td>
<td>0.3</td>
<td>1440</td>
<td>2.90</td>
<td>25</td>
</tr>
<tr>
<td>FRP-2</td>
<td>191000</td>
<td>0.3</td>
<td>2880</td>
<td>1.45</td>
<td>25</td>
</tr>
</tbody>
</table>

**Modeling**

In this paper, steel panel and FRP plates were modelled by 3D zones in BEM software BEASY, and internal spring was applied to simulate the adhesive layer. Stiffness of internal spring was determined by Young’s modulus, Poisson ratio and thickness of adhesive. Figure 1 and Figure 2 show the dimensions of steel panel and patching configuration. The width of FRP plates is 25 mm. Half of the specimens was modelled due to the symmetry of the geometry and boundary conditions. Then, only the boundary of zones need be meshed in BEM, and quadratic element was selected. Rigid body displacement was constrained by boundary spring shown in Figure 3. In this way, the displacement field near crack tip could be restricted, but it would not have a significant impact on the stress field near crack tip.

Seven models were simulated using BEM as shown in Table 3. All of them were subjected to cyclic tensile load with a stress ratio of 0.1, and a constant stress range of 153 MPa.

**COMPARISON OF CRACK GROWTH CRITERIA**

The influence of crack growth criterion was studied with U-MCSC and U-MSEDC. Figure 4 plotted crack length and fatigue cycles curves of U-MCSC and U-MSEDC. Crack length $S$ is defined in Eq.(4). The fatigue lives of U-MCSC and U-MSEDC are 111497 and 110494, respectively. Deviation between them is only 0.85%. Figure 5
presents the crack paths of two models in Cartesian coordinates that origin is the center of steel panel. We can find that the two paths are almost same.

\[ S = \int \sqrt{dx^2 + dy^2} \]  

(4)

Table 9 List of models

<table>
<thead>
<tr>
<th>No</th>
<th>Patch configuration</th>
<th>Prestress(%)</th>
<th>Crack growth criterion</th>
</tr>
</thead>
<tbody>
<tr>
<td>U-MCSC</td>
<td>Unrepaired</td>
<td>—</td>
<td>MCSC</td>
</tr>
<tr>
<td>U-MSEDC</td>
<td>Unrepaired</td>
<td>—</td>
<td>MSEDC</td>
</tr>
<tr>
<td>R-MSEDC-0-(1.45)</td>
<td>Repaired by FRP-2</td>
<td>0</td>
<td>MSEDC</td>
</tr>
<tr>
<td>R-MSEDC-0</td>
<td>Repaired by FRP-1</td>
<td>0</td>
<td>MSEDC</td>
</tr>
<tr>
<td>R-MSEDC-5</td>
<td>Repaired by FRP-1</td>
<td>5</td>
<td>MSEDC</td>
</tr>
<tr>
<td>R-MSEDC-10</td>
<td>Repaired by FRP-1</td>
<td>10</td>
<td>MSEDC</td>
</tr>
<tr>
<td>R-MSEDC-15</td>
<td>Repaired by FRP-1</td>
<td>15</td>
<td>MSEDC</td>
</tr>
</tbody>
</table>

Figure 4 Number of fatigue cycles vs. crack length S

Figure 5 Crack path

Figure 6 shows the relationship between crack length and SIF. It reveals that the values of SIF are almost same, when crack length is less than 23 mm. Then, \( K_I \) of the model U-MCSC has a sudden drop. This corresponds to the fluctuation of \( K_{II} \), and also leads to the difference of crack paths. However, when the crack length is 23mm, the number of fatigue cycles reaches about 95% of fatigue life. So, only MSEDC was applied in the following simulations.

![Comparison of FRP-1 and FRP-2](image)

COMPARISON OF FRP-1 AND FRP-2

FRP-1 and FRP-2 were used to strengthen the cracked steel plates. The tensile stiffness of FRP-1 plate and FRP-2 plate was same, as expressed in Eq.(5), where \( E_1 \) and \( E_2 \) are Young’s modulus of FRP-1 and FRP-2, \( t_1 \) and \( t_2 \) are thickness of FRP-1 and FRP-2.

\[ E_1 \cdot t_1 = E_2 \cdot t_2 \]  

(5)

Figure 7 shows the relation curves between fatigue cycles and crack length of R-MSEDC-0-(1.45) and R-MSEDC-0. The fatigue life of R-MSEDC-0-(1.45) is 169277. It is 1.5% more than the fatigue life of R-MSEDC-0. This is
because of that FRP material near bottom surface makes more contribution to restricting crack opening than that near top surface. Liu et al. also concluded similar result. They found that increasing thickness of the composite patch beyond a certain minimum value dose not result in any significant increase in fatigue life (LIU et al. 2012).

Values of SIF are graphed in Figure 8, where we can find that two curves coincide with each other. From this point of view, strengthening effect of FRP-1 plate is equivalent to that of FRP-2. However, thinner FRP plate caused more element numbers and longer computing time. Therefore the other steel panels were all patched with prestressed FRP-1 plates.

![Figure 7 Fatigue cycles vs. crack length S](image1)

![Figure 8 Crack length S vs. values of SIF](image2)

**EFFECTS OF PRESTRESSED FRP PLATES ON STEEL PANELS FATIGUE BEHAVIOR**

Values of SIF during the crack propagation of U-MSEDC, R-MSEDC-0, R-MSEDC-5, R-MSEDC-10 and R-MSEDC-15 are shown in Figure 9. It indicates that patching FRP plate can decrease $K_I$ dramatically. The values of $K_I$ will become smaller as prestress level increases. When the crack of steel panel, patched with prestressed FRP, keeps open during fatigue loading, the positive part of $\Delta K_I$ will not be changed with prestress. In this case, prestress will not enlarge the fatigue life of steel panel with Paris law and Tanaka formula. From Figure 9(b), we can find that when fatigue load decreases to 17 MPa, the values of $K_I$ are still positive. So, fatigue life of R-MSEDC-5 is not much different from that of R-MSEDC-0.

![Figure 9 Effect of prestressed FRP plates on SIF](image3)

![Figure 10 Fatigue cycles vs. crack length S](image4)
Figure 11 Fatigue life contributed by different part of model

Curves between crack length and fatigue cycles of these models are compared in Figure 10. Both patching and prestressing can improve fatigue performance of steel panels with inclined crack. Figure 11 shows the fatigue life contributed by FRP and prestress. The fatigue life of R-MSEDC-0 and R-MSEDC-5 are 166674 and 168787. Deviation between them is 1.25%. Fatigue life of R-MSEDC-0 has a 51% increase versus that of U-MSEDC. Fatigue life of R-MSEDC-10 and R-MSEDC-15 have 23.3% and 62.8% increase versus that of R-MSEDC-0. So we can conclude that the prestress increase will accelerate fatigue performance improvement. High prestressed FRP plate is recommended to repair cracked steel panel.

CONCLUSIONS

In this paper, crack propagation in steel panels patched with prestressed and non-prestressed FRP plates is simulated by BEM. From the results, we can conclude that:

1. When mode I crack plays a dominant role during crack growth, there will not be much difference between MSEDC and MCSC.

2. The value of positive part of $\Delta K_I$ is diminished after repairing with FRP plate. So that a longer fatigue life can be achieved.

3. Fatigue life of steel panel patched with non-prestressed FRP plate has a 51% increase versus that of unrepaired steel panel. Whereas, there is little difference between steel panels patched with non-prestressed and 5% prestressed FRP plate.

4. Fatigue life improvement can be increased quickly with increment of prestress. Therefore, high prestressed FRP plate is suggested.

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steel plates”, *Composite Structures*, 123, 109-122.


Mini-symposium on Strengthening of Steel Structures with FRP

Organizers:
Jun DENG
Yongxin YANG
DURABILITY OF STEEL BEAMS STRENGTHENED WITH CFRP PLATE

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ABSTRACT

CFRP external bonding is an innovative technique to repair or strengthen steel beams. The weakest link of the retrofitted beams is the adhesive bonding, and plate peeling is the most important failure mode due to the high stress concentration at the discontinuous sections. Adhesive bonding subjected to fatigue loading and hygrothermal environment may have a significant adverse effect. This paper presents a study on the durability of the steel beams strengthened with CFRP plate. Firstly, a fatigue test programme of the retrofitted steel beams was conducted. An $S-N$ curve was proposed to predict the crack-free fatigue life of the retrofitted beams. An equation to predict the crack-propagation fatigue life had also been developed. Secondly, the behaviour of overloading damage steel beams strengthened with CFRP plate exposed to hygrothermal environment was investigated. The accelerated ageing experiments were carried out for 3 months and 6 months by exposing specimens to cyclic wetting in salt water and drying in hot air. The specimens were tested to failure under four-point bending. The results show that the hygrothermal environment weakened the bonding behaviour of the adhesive.

KEYWORDS

CFRP, steel beams, strengthening, durability.

INTRODUCTION

Highway bridges are designed for lives of more than 100 years. Depending on the type of highway, components in decks can experience up to a hundred million stress cycles over their design lives (May and Tilly, 1982, Moses et al., 1987). Consequently, fatigue damage is one of the main problems that occur in old metallic bridges. Hollaway and Cadei (2002) pointed out that the adhesive bonding between the CFRP plate and the metallic substrate is the weakest link and its fatigue performance is of particular importance. The stress concentrations conduct fatigue interfacial failure at the plate ends (Deng and Lee, 2007a) or at the crack locations (Colombi and Fava, 2015). Colombi and Fava (2015) indicated that reinforcement debonding has a detrimental effect on the reinforcement effectiveness and it lessens the fatigue life. Moreover, it is critical to understand the influence of the environmental conditions on the behaviour of the adhesive bonding. Kim et al. (2012) found an increase in the load carrying capacity of steel-CFRP joints after wet/dry cycles and freeze/thaw cycles. However, Nguyen et al. (2012) observed about 10% reduction in the bond strength after exposed to thermal cycling. Agarwal et al. (2014) present that the bond strength of steel-CFRP joint reduced 28% after freeze/thaw cycles. In this paper, a study on the durability of the steel beams strengthened with CFRP plate was conducted. Fatigue life of the retrofitted steel beams and the effects of overloading damage and hygrothermal environmental exposure were investigated.

Figure 1 Details of retrofitted beam
THEORETICAL STUDY

The details of the retrofitted beams were shown in Figure 1. The maximum interfacial stresses of the steel beams strengthened with CFRP plate are given in this section. An empirical equation to predict fatigue life during the crack propagation is developed based on Paris Law as well.

Maximum interfacial stresses

At the boundaries, the maximum shear stress \( \tau_{\text{max}} \) can be written as:

\[
\tau_{\text{max}} = \frac{1}{b} \lambda c - \frac{Z_s}{bf} E I V(0)
\]  

(1)

where

\[
\lambda = \sqrt{\frac{f_s}{f_t}}
\]  

(2)

\[
f_t = \frac{t_a}{Gb}
\]  

(3)

\[
f_s = \frac{(Z_s + Z_f)Z_t}{E f I_t} + \frac{1}{E_s A_s} + \frac{1}{E_f A_f}
\]  

(4)

\[
c = N_{f0} - \frac{\Delta \varepsilon_{sf}}{f_s}
\]  

(5)

\[
\Delta \varepsilon_{sf} = (\alpha_f - \alpha_s) \Delta T \frac{M_s Z_s + F_p}{E_s A_s} + \frac{M_f Z_f}{E_f I_f}
\]  

(6)

\( \Delta \varepsilon_{sf} \) is the relative deformation between the bottom of the steel beam and the top of CFRP plate when the adhesive layer is ignored, including the relative deformations caused by temperature change, load-release jacking, prestressing force and applied moment. The subscripts \( s, f \) and \( a \) denote steel beam, CFRP plate and adhesive, respectively. \( Z_s, Z_t, b \) and \( t_a \) the distance from the neutral axis to the bottom of the steel beam, the distance from the neutral axis to the top of the CFRP plate, the plate width and the adhesive thickness. \( E, I \) and \( A \) are the elastic modulus, the second moment of area and the area, respectively. \( M_s, V(0) \) and \( N_{f0} \) are the applied bending moment, the applied shear force and the longitudinal force at the boundaries, respectively. \( \alpha \) and \( \Delta T \) are the thermal expansion coefficient and the temperature change, respectively. \( M_p \) is the bending moment applied on the steel beam by load-release jacking. \( F_p \) is the prestressing force applied on the CFRP.

At the boundaries, normal stress \( \sigma_{\text{max}} \) can be written as:

\[
\sigma_{\text{max}} = \left( \beta - \frac{\lambda}{2} \right) \frac{t_a}{b} c - \frac{\beta f_s Z_s}{bf} E I V(0)
\]  

(7)

where

\[
\beta = \sqrt{\frac{f_s}{4 f_s}}
\]  

(8)

\[
f_s = \frac{t_a}{E_f b}
\]  

(9)

\[
f_s = \frac{1}{E_f I_f} + \frac{1}{E_s I_s}
\]  

(10)

Combining the maximum shear and normal stresses, the maximum tensile principal stress \( \sigma_{1\text{max}} \) can be written as:

\[
\sigma_{1\text{max}} = \frac{\sigma_{\text{max}}}{2} + \sqrt{\left( \frac{\sigma_{\text{max}}}{2} \right)^2 + \tau_{\text{max}}^2}
\]  

(11)

Fatigue life prediction

The fatigue life includes crack-free life and crack-propagation life. Crack-free life can be obtained from the S-N Curve. In accordance with the FE analysis (Deng and Lee, 2008), the crack will initiate in the adhesive from the end of the plate and propagate along the interface between the steel beam and adhesive. In accordance with Paris Law, the relationship between the crack propagation rate \( da/dN \) and the energy release rate \( G \) can be expressed as:
\[
\frac{da}{dN} = D(G_{\text{max}})^n
\]

where \(G_{\text{max}}\) is the maximum energy release rate during one fatigue cycle, \(D\) and \(n\) the empirical coefficients, which can be determined by experimental data. Guidance CIRIA C595 (Cadei et al., 2004) provides an equation to calculate the energy release rate \(G\) of strengthened beams:

\[
G = \frac{M^2}{2b} \left[ \frac{1}{(EI)_b} - \frac{1}{(EI)_s} \right]
\]

where \(M\) is the applied bending moment on the beam at the plate end or the crack front, \((EI)_b\) and \((EI)_s\) the section bending stiffness of beam strengthened with the FRP plate and the plain beam, respectively.

The crack propagation life \(N\) of the retrofitted beams can be obtained as:

\[
N = \frac{(-2n+1)^{-1}(l_0 + a)^{-2n+1} + C}{D\left[ \frac{1}{4} P^2 \left( \frac{1}{(EI)_b} - \frac{1}{(EI)_s} \right) \right]}
\]

where

\[
C = (-2n + 1)^{-1}(l_0 + a_0)^{-2n+1}
\]

\[
a_0 = \frac{E_G}{\pi \sigma_n^2}
\]

EXPERIMENTAL STUDY

To investigate the durability of the retrofitted steel beams, a fatigue experiment study was conducted to obtain the \(S-N\) curve and \(a-N\) curve. Moreover, an experimental study on overloading damaged retrofitted steel beams subjected to wet-dry environmental exposure was conducted.

Fatigue tests

A minimum load of 5 kN was applied to all the fatigue tested beams so as to ensure firm contact between the beam and the supports. The maximum loads are from 40 kN to 125 kN. Loading was applied sinusoidally, with a frequency of 1 to 2 Hz. During the fatigue cyclic loading, the test was stopped at regular intervals, or when cracking was detected, and the specimen was then subjected to three cycles of loading and unloading between 0 kN and the maximum applied load at a rate of 2 kN/second. This was done in order to measure the loads, the strain values at the CFRP plate ends and the deflections during the load cycle, since these data cannot be recorded accurately during the fatigue load cycles.

Overloading damaged tests

The specimens were damaged under overloading fatigue loading. Loading was applied sinusoidally, with a frequency of 0.1 Hz. The minimum and maximum cycle loads are 0.1\(P_c\) and 0.5\(P_c\), where \(P_c\) is the loading when crack initiates in the bonding layer. From the Miner’s rule and the overloading spectrum (Wang et al., 2014), the fatigue damage accumulation index for the retrofitted steel beams per year is \(28.5 \times 10^4\) and thereby the number of cycles in the tests is 400.

Wet-dry environmental exposure

The wet-dry environment was produced by placing specimens into salt water under wetting/drying cycles. The wetting/drying environmental chamber with a length of 3 m, a width of 2 m and a depth of 1.5 m can automatically adjust the water content. The temperature control accuracy is \(\pm 1^\circ\)C. Each wetting/drying cycle was 24 h. The specimens were immersed in a 3.5% NaCl solution for 10 h, following by drying at 40 \(^\circ\)C for 14 h. The specimens were exposed for 3 months and 6 months, respectively.

Static tests

The tests were carried out in a servo-hydraulic SDS500 test machine with a maximum capacity of 500 kN, subjected to a four-point bending set up, as shown in Figure 2. The specimens were tested under static load, by displacement control at a rate of 0.05 mm/sec. Loading of the specimen continued even after the CFRP plate peeling, which occurred in all retrofitted specimens. Loading was stopped when local post-yield buckling appeared in the control specimen or when the notched specimens fractured with the notch propagation.
RESULTS AND DISCUSSIONS

S-N curve and crack-free life

The maximum principal interfacial stresses ($\sigma_{\text{1max}}$) of all specimens are calculated by Eq. (11). The curves for $\sigma_{\text{1max}}$ versus the log of the crack free life $N_1$ is presented in Figure 4. $\sigma_{\text{1max}}$ (80.4 MPa) is the maximum principal interfacial stresses of specimen with a maximum applied load of 135 kN. The relationship is approximately linear and regression analyses produce the following best-fit equation (with a correlation coefficient, $R^2 = 0.96$):

$$\sigma_{\text{1max}} = -4.19Ln(N_1) + 78.62$$  \hspace{1cm} (17)

It has been established by the authors that the bonded strength of retrofitted metallic beams is not influenced by the size and material properties of the metallic beams or the CFRP plates (Deng and Lee, 2007b). Therefore, this S-N curve can be applied generally as long as the same adhesive is used. The fatigue threshold for the tested beams is 40 kN, with a corresponding threshold stress 23.8 MPa for the adhesive. This threshold limit is about 30% of the ultimate failure stress 80.4 MPa under static loading.

Crack propagation life

The crack initiation and propagation in all the specimens were similar. For all the plates that had debonded, cracking started from the middle of the spew fillet and then propagated to the interface between the steel beam and the adhesive at an angle of 45 degrees. Then the crack grew along the interface and stopped eventually. There was always a short length of adhesive remaining uncracked that bonded the plate to the beam. The curves for the log of the crack propagation rate $da/dN$ versus the log of the energy release rate $G$ is presented in Figure 5. The
relationship is approximately linear and regression analyses produce the following best-fit equation (with a correlation coefficient, $R^2 = 0.7896$):

$$\log(da/dN) = 2.753\log(G_{\text{max}}) - 11.718$$

Therefore, the empirical coefficients $D$ and $n$ in Eq (12) are $10^{-1.7}$ and 2.75, respectively.

**Strength of the specimens after environmental exposure**

The load versus displacement plots of the specimens are shown in Figure 6. Two aspects are noted from Figure 6. Firstly, the stiffness and strength of the reference beam AR is higher than the beams ARFH which were overloading damaged and subjected to dry-wet cycles. Secondly, the stiffness and strength of the specimen ARFH-90 subjected 90 days dry-wet cycles are higher than ARFH-180, which indicates that the retrofitted steel beams deteriorated with the period of the dry-wet cycles.

The debonding loads of all specimens were compared in Figure 7. It showed that the beams subjected to overloading fatigue tests have lower debonding loads than others. The debonding loads of the specimens reduced with the days of the dry-wet environmental exposure.

**Failure modes of the specimens after environmental exposure**

The failure modes of the strengthened notched beams were the CFRP plate peeling but part of it remained attached to the beam. The typical failed specimen was showed in Figure 3. Figure 8 shows the bonded surfaces of the CFRP plate and steel beams of the failed specimens which the CFRP plates were detached totally from the steel beam by hand. The figure shows that debonding in specimens AR and ARFH developed from the notch in the steel beams along the interface between the CFRP plate and the adhesive. In specimen ARH1-180 subjected to the dry-wet cycles, however, the corrosion of the steel caused the debonding initiated along the interface between the steel beam and the adhesive. In specimen ARFH-180, the debonding still initiated along the interface between the CFRP plate and the adhesive, which indicates that the overloading fatigue damaged this interface where the steel corrosion stain can be observed.
CONCLUSIONS

This paper presents a study on the durability of the steel beams strengthened with CFRP plate. Fatigue life of the retrofitted steel beams and the effects of overloading damage and hygrothermal environment exposure were investigated. The accelerated ageing experiments were carried out for 3 months and 6 months by exposing specimens to cyclic wetting in salt water and drying in hot air. From the fatigue tests results, an S-N curve is proposed to predict the crack-free fatigue life of the retrofitted beam. An equation to predict the crack-propagation fatigue life has also been developed. The test results show that overloading fatigue damage reduced the debonding loads of the retrofitted beams subjected to the environmental exposure. Moreover, the debonding loads of the specimens reduced with the days of the dry-wet environmental exposure.

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EXPERIMENTAL STUDY ON INTERFACIAL BONDING BEHAVIORS BETWEEN ECC AND CONCRETE

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ABSTRACT
In this paper, an experimental study was conducted to investigate the bonding behaviors of engineering cementitious composite (ECC)-to-concrete interface. Considering the construction method of ECC and cured interface, a total of 18 specimens were prepared. All specimens were prism with the dimension of 40mm (width) × 80mm (depth) × 160mm (length) and consisted of two parts: ECC layer with an approximately thickness of 40 mm on the top and the concrete substrate at the bottom. The bonding area for all specimens was about 40mm (width) × 160mm (length). Three types of interfaces were obtained by cured the top surface of concrete substrate: type A was simply only removing the cement paste attached to the concrete substrate and then cleaning the surface of bond substrate; type B was spreading immediately 0.5mm-1.75mm fine aggregate followed by a 2mm thickness epoxy resin layer brushed and type C was similar with type B additionally applying 2.5mm-5mm coarse aggregate. After the concrete substrates were cured 28 day, the casting ECC and spraying ECC were placed directly against this to form the bonding interface. The pull-out force acted on the bonding interface was implemented by a universal tensile machine. The test results showed that the interfacial full debonding occurred for specimens with type A and B interfaces, while the fracture failure was observed in the ECC layer for specimens with type C interface. The roughness degree induced by different type of interface had a significant influence on the bond strength of ECC-to-concrete interface and coarser interface can cause higher interfacial bond strength. Meanwhile, the interfacial bond strength was decreased by application of the sprayed ECC for all three types of interface. It is concluded that the interfacial bond performance can be improved effectively by treating the bond interface.

KEYWORDS
Engineered cementitious composites (ECC), concrete, interface, bond strength.

INTRODUCTION
The engineered cementitious composites (ECC) has many advantages, such as ultra high toughness (Ahmed et al. 2009; Kim et al. 2011), multiple micro-cracking behaviors (Wang et al. 2016), self-healing characterization (Suryanto et al. 2015), better fatigue resistance (Leung et al. 2007; Qian et al. 2013) and good durability (Yildirim et al. 2015). One of important application for ECC is repairing or rehabilitating the damage structures, such as bridge decks, main girders and tunnel linings (Zheng et al. 2016). Therefore, the interfacial properties between ECC and old concrete are the most critical and then it need to be studied deeply (Wang et al. 2016).

As so far, very limited research work has been conducted to investigate the influences of concrete and ECC strengths (Bu et al. 2012), roughness degree of concrete substrate (Bu et al. 2012; Carbonell et al. 2013), fiber types (Kamal et al. 2014) and curing time of interface (Carbonell et al. 2013) on the bonding behaviors of ECC-to-concrete interface and the fatigue performance of the ECC-to-concrete interface (Zhang and Li 2002; Liu et al. 2013). Usually, the casting ECC was used in their researches and then the influence of construction method for this material on the interfacial behaviors was not considered. Theretorefore, the casting and spraying ECC were both applied in this paper to investigate the effect of contortion method on the bonding behaviors of ECC-to-concrete interface. Moreover, different cured interface between the concrete and ECC was also included.
EXPERIMENTAL STUDY

Description of specimens

A total of 18 specimens were prepared to investigate the bonding behaviors of ECC-to-concrete interface. All specimens have a prism shape with the designed geometric dimensions of 160mm in length, 80mm in depth and 40mm in width. The ECC layer with an approximately thickness of 40 mm on the top and the concrete substrate at the bottom were bond together by a 40mm × 160mm bond interface, as shown in Figure 1.

All specimens are divided into three groups and each group has three identical coupons according to the construction method of ECC and type of interface. Specimen designation is as follows and is described in Table 1. The initial letter (N or S) indicates the construction method of ECC (e.g. N is casting ECC and S is spraying ECC). The second letter (A, B or C) represents the interface type. Type A is simply only removing the cement paste attached to the concrete substrate and then cleaning the surface of bond substrate. Type B is spreading immediately 0.5mm-1.75mm fine aggregate followed by a 2mm thickness epoxy resin layer brushed and finally Type C is similar with type B additionally applying 2.5mm-5mm coarse aggregate, as shown in Figure 2.

Before manufacturing the specimens, a steel mould was manufactured. The steel mould was consisted of two-compartment, one for the concrete substrate and another for ECC. To enhance the interfacial bonding performance, two steel rivets were welded on one steel plate and pre-embedded into the concrete and ECC. Moreover, a steel rebar was welded on another face of that steel plate so that it can be clamped by the chucks of tensile machine, as shown in Figure 1.

The process of making specimens are as follows: 1) casting concrete into one steel compartment to form the substrate; 2) removing the surface cement paste at the substrate concrete interface after the substrate concrete was cured 28 day; 3) treating the interface according to interfacial type as mentioned above; 4) casting or spraying ECC into another steel compartment against the cured interface directly to form the specimen, as shown in Figure 1. All the specimens were tested after 28 days curing of the ECC under the temperature of 20±2°C and relative humidity of 95%.

Figure 1 Specimens: (a) front view; (b) side view

Figure 2 Interface types A, B and C
Table 1 Summary of test specimens and experimental results

<table>
<thead>
<tr>
<th>Specimen types</th>
<th>Geometric dimensions of ECC-concrete interface (mm)</th>
<th>Numbers of specimens (group × number)</th>
<th>Interface type</th>
<th>Construction method of ECC</th>
<th>Maximum tensile load (kN)</th>
<th>Nominal interfacial strength (MPa)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>N-A</td>
<td>40×160</td>
<td>1×3</td>
<td>Type A</td>
<td>Casting</td>
<td>1.856</td>
<td>0.29</td>
<td>Interfacial debonding</td>
</tr>
<tr>
<td>N-B</td>
<td>40×160</td>
<td>1×3</td>
<td>Type B</td>
<td>Casting</td>
<td>3.136</td>
<td>0.49</td>
<td>Interfacial debonding</td>
</tr>
<tr>
<td>N-C</td>
<td>40×160</td>
<td>1×3</td>
<td>Type C</td>
<td>Casting</td>
<td>4.416</td>
<td>0.69</td>
<td>ECC fracture</td>
</tr>
<tr>
<td>S-A</td>
<td>40×160</td>
<td>1×3</td>
<td>Type A</td>
<td>Spraying</td>
<td>0.832</td>
<td>0.13</td>
<td>Interfacial debonding</td>
</tr>
<tr>
<td>S-B</td>
<td>40×160</td>
<td>1×3</td>
<td>Type B</td>
<td>Spraying</td>
<td>2.752</td>
<td>0.43</td>
<td>Interfacial debonding</td>
</tr>
<tr>
<td>S-C</td>
<td>40×160</td>
<td>1×3</td>
<td>Type C</td>
<td>Spraying</td>
<td>3.904</td>
<td>0.61</td>
<td>ECC fracture</td>
</tr>
</tbody>
</table>

Materials

The ECC mixtures were consisted of cement, fly ash, silica fume, water, water-reducing agent, accelerating agent, silica sand and polyvinyl alcohol (PVA) fibers. Ordinary Portland cement was applied. The sizes of silica sand were from 0.15mm to 0.32 mm. The 39 μm diameter and 12 mm length PVA fibers manufactured by Kuraray Corporation of Japanese were selected to reinforce the cement base. The PVA fibers added to the matrix were 2% of total material volume. The measured 28-day compressive strengths of ECC and concrete substrate were 32.3 MPa, and 43.2 MPa, respectively. The detailed mixture proportions of concrete and ECC are shown in Table 2.

Table 2 Mixture proportions of the substrate concrete and ECC (kg/m³).

<table>
<thead>
<tr>
<th>Mixture type</th>
<th>Water</th>
<th>Cement</th>
<th>Fly ash</th>
<th>Silica fume</th>
<th>Fine aggregate</th>
<th>Coarse aggregate</th>
<th>Water-reducing agent</th>
<th>Accelerating agent</th>
<th>Fiber volume fraction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>180.0</td>
<td>450.0</td>
<td>-</td>
<td>-</td>
<td>530.0</td>
<td>1200.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>ECC</td>
<td>473.0</td>
<td>956.5</td>
<td>286.5</td>
<td>40.0</td>
<td>250.0</td>
<td>-</td>
<td>1.0</td>
<td>20.0</td>
<td>2%</td>
</tr>
</tbody>
</table>

Test setup

The uniaxial pull-out force acted on the bonding interface was implemented by a universal tensile machine, as shown in Figure 3. The whole loading process was controlled by displacement and the speed of displacement was 0.2 mm/min. Displacement and loads were real-time collected by an automatic data acquisition system.

![Figure 3 Test setup.](image-url)
RESULTS AND DISCUSSION

Failure modes

Two failure modes were observed in the test. For specimens with Type A and B interfaces, the interfacial full debonding was observed, as shown in Figure 4. For specimens with Type C interface, ECC fracture along the bond interface occurred, as shown in Figure 5. Although the construction method of ECC is different for specimens in Group N and Group S, the failure mode has not changed for different type of interface. It can be concluded that the roughness degree of concrete substrates has a significant influence on the failure mode, while the casting method of ECC almost has no obvious influence on the failure mode.

The load-displacement curves of one selected specimen from the corresponding group under the uniaxial pull-out load are shown in Figure 6. It can be seen form Figure 6, the load-displacement responses exhibit nearly a linear relationship before the applied loads reach to the maximum values. When applied load beyond the peak values, the curves drop straightly until the specimens failed.

The experimental results were summarized in Table 1. Compared with the type A interface specimens in Group N-A, the interfacial debonding strength of specimens with types B and C of interfaces are increased significantly. Similar situation can be also seen in groups S-A, S-B and S-C using the spraying ECC. For example, the mean values of interfacial debonding strength are only 0.29 MPa for specimens in group N-A, but a higher value (i.e. 0.49 MPa for group N-B and 0.69 MPa for group N-C) can be obtained by using the coarse interface. Therefore, the roughness degree of concrete substrates had a significant influence on the interfacial debonding strength between ECC and concrete. In addition, it can also found that the nominal interfacial debonding strength for the spraying ECC were all smaller than that for the casting ECC no matter what types of interface are applied, which indicates the influence of construction method of ECC was obvious on the interfacial debonding strength.

\[
\tau = \frac{F_{\text{max}}}{A}
\]

where \(\tau\) is nominal interfacial debonding strength, MPa; \(F_{\text{max}}\) is maximum pull-out load, kN; \(A\) is interfacial area, mm\(^2\).

Nominal interfacial debonding strength
CONCLUSIONS

The effect of contortion methods of ECC and interface types on the bond behaviors of ECC-to-concrete interface were experimentally investigated in this paper. Two failure modes were observed during the test, which were the interface debonding for specimens with type A and B interfaces and ECC fracture for specimens with type C interface. The cured interface had a significant influence on the failure mode and the debonding strength of interface and coarser interface can cause higher interfaceal bond strength, while the construction methods of ECC only affect the interfacial bond strength. Relatively low bond strength was obtained for specimens manufactured with spraying ECC no matter what types of interfaces was applied in the test specimens. It is concluded that the influence of construction method of ECC was obvious on the interfacial debonding strength.

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REFERENCES


ANALYSIS OF BUCKLING BEHAVIORS OF CFRP-ALUMINUM HYBRID COLUMNS

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ABSTRACT

Combining Al (namely, aluminum alloy) and CFRP (carbon-fiber-reinforced polymer) jackets to form hybrid columns can yield many advantages. For stub ones, bearing capacity can be improved significantly. For long ones, hybrid columns have improved buckling behaviors compared with traditional pure Al columns. Firstly, the elastic buckling bearing capacity based on the margin yield criterion of Al will increase. Secondly, the maximum buckling bearing capacity based on maximum strength criterion will also increase. Thirdly, the deformation capacity will improve. In this paper, for stub specimens, a systematic study of CFRP-Al hybrid columns is conducted including experiments, theoretical analysis and finite element analysis (FEA). Specially, for long specimens, due to the fact that current theoretical model is not suitable for hybrid specimens, FEA is used to get the maximum buckling bearing capacity of them. Besides, typical column curves for long hybrid specimens are obtained by FEA based on maximum strength criterion for design.

KEYWORDS

Aluminum alloy, CFRP jacket, local buckling, global buckling, margin yield criterion, finite element analysis.

INTRODUCTION

A combination of composites with other structural materials, such as steel (Feng et al. 2013), concrete (Feng et al. 2015), and natural materials (Feng et al. 2012), can be created to yield favorable structural behaviors. Instead of simply replacing other materials, this method enables the direct and efficient use of composites in construction. Many studies have focused on FRP columns and FRP-metal hybrid columns (Guades et al. 2013; Han et al. 2007; Jiao et al. 2004), which have identified new possibilities for combining FRPs and metals for structures in construction and have demonstrated improved properties compared to traditional structures. Al and FRP have many similar advantages like low weight, good corrosion resistance, free maintenance, no ferromagnetic properties and flexible manufacturing procedures (Ye and Feng 2006; Zhao and Zhang 2007). CFRP-Al hybrid components retain these advantages and also compensate for some of their disadvantages. Previous studies have demonstrated considerable interest in reinforcing Al structures with FRPs (Kim 1998; Lee et al. 2004; Zhang et al. 2014). This paper presents experimental and theoretical investigations of the compressive capacities and buckling behaviors of stub and long CFRP-Al hybrid columns.

EXPERIMENT

The mechanical properties of the Al materials and CFRP and the specimen number are the same as those in the previous paper (Feng et al. 2016). The section configurations are shown in Figure 1.
The test results of short specimens are shown in the previous paper (Feng et al. 2016). For long specimens, due to the fact that the CFRP jacket will not yield, the hybrid long specimens have the following global buckling behaviors. Figure 2 shows the typical force-lateral displacement curves of long specimens, which indicates the tendency. Firstly, the elastic buckling bearing capacity \( F_A \) based on the margin yield criterion of Al will increase. Secondly, the maximum buckling bearing capacity \( F_B \) based on maximum strength criterion will also increase. Thirdly, the index of judging deformation capacity can be defined as the lateral displacement when the load decreases to 85% of its maximum load (Feng et al. 2013). \( \Delta_{\text{hybird}} \) and \( \Delta_{\text{Al}} \) represent the lateral displacement of hybrid columns and Al pure columns, respectively. It is obvious that \( \Delta_{\text{hybird}} \) is much larger than \( \Delta_{\text{Al}} \). Thus, it is seen that the deformation capacity after bucking will also improve.

\[
\begin{align*}
\Delta_{\text{hybird}} & > \Delta_{\text{Al}} \\
\text{Thus, it is seen that} \quad \text{the deformation capacity after bucking will also improve.}
\end{align*}
\]

DISCUSSION OF THEORETICAL MODEL FOR \( F_B \)

For stub columns, local buckling or strength failure may happen. Based on calculation method proposed by Feng et al. (Feng et al. 2016), \( F_B \) can be obtained accurately.

For long columns, global buckling happened. In order to get \( F_B \), the stability coefficient \( \varphi \) should be obtained. Based on the margin yield criterion, Eq. 1 gives the way to calculate \( \varphi \), which can also be called Perry-Robertson formula.

\[
\varphi = \frac{1}{2 \lambda^2} \left[ (1 + \epsilon_0 + \lambda^2) - \sqrt{(1 + \epsilon_0 + \lambda^2)^2 - 4 \lambda^2} \right] 
\]  
(1)

In the “Technical code of cold-formed thin-wall steel structures” (GB50018-2002), Eq. 1 has also been adopted to obtain \( \varphi \). Specifically, the equivalent relative initial bending \( \epsilon_0 \) can be obtained by the following equations, where \( \lambda \) is the regularized slenderness ratio:

\[
\begin{align*}
\bar{\lambda} \leq 0.5, \quad \epsilon_0 &= 0.25 \bar{\lambda} \\
0.5 < \bar{\lambda} \leq 1.0, \quad \epsilon_0 &= 0.05 + 0.15 \bar{\lambda} \\
\bar{\lambda} \geq 1.0, \quad \epsilon_0 &= 0.05 + 0.15 \bar{\lambda}^2
\end{align*}
\]  
(2-4)

However, the mechanism of hybrid long columns is different from pure ones. Figure 3 shows the change of the stress distribution of the hybrid columns during loading. Due to the high strength of CFRP, the margin will not

\[
\begin{align*}
\Delta_{\text{hybird}} & > \Delta_{\text{Al}} \\
\text{Thus, it is seen that} \quad \text{the deformation capacity after bucking will also improve.}
\end{align*}
\]
yield. Thus, the Perry-Robertson formula is only available for pure columns. Besides, a simple theoretical way is hard to get $F_B$. Therefore, FEA is used in the following analysis to get $F_B$.

**Figure 3** Change of the stress distribution of long hybrid columns during loading

**FEA**

FE modeling was performed by using ANSYS software (Díaz C et al. 2011; Yang et al. 2013); the Al was meshed by using the 3-D structural solid element SOLID45, and the CFRP jacket was meshed by using the layered 3-D structural solid element SOLID46.

**Stub columns**

The arc length method was applied to solve nonlinear problems. The values of the compressive bearing capacity are in good agreement of experimental results (Feng et al. 2016).

**Long columns**

No debonding was observed in the long columns; thus, the nodes that were shared by the CFRP jacket and the Al were coupled. The FEA was conducted as the followings: eigenvalue buckling analysis was conducted thus the first-order buckling mode was obtained. Then non-linear buckling analysis was conducted. The good results are shown in Figure 4, where $\phi_{FEM}$ and $\phi_E$ are stability coefficients from FEM and experiment, respectively.

**Figure 4** Comparison of stability coefficient $\phi$ from experiment and FEM
Based on the reliable FE modeling, the column curves can be obtained. Figure 5 shows the relationship of $\varphi$ and $\lambda$. It is obvious that long hybrid columns can be divided into two categories: $\lambda \geq 1.25$ and $\lambda < 1.25$.

**Column curve of Specimens of $\lambda \geq 1.25$**

$\xi$ is defined as the ratio of the bending stiffness that is provided by the CFRP jacket to that of the entire specimen, as shown in Eq. 5:

$$\xi = \frac{(\sum E_{L,C} I_{C})}{(\sum E_{L,C} I_{C} + E_{A} I_{A})}$$  \hspace{1cm} (5)

Based on Eqs 2 to 4, the value of $\omega_0$ is influenced by $\xi$ and $\lambda$. The values of $a_1$ and $a_2$ are defined as initial imperfection factors based on $\xi$. The values of $a_1$ and $a_2$ could be obtained from the relationship of $\varphi$ and $\lambda$ by Eq. 7 at different $\xi$. The least square method was used to get the relationship of $a_1$ and $\xi$, $a_2$ and $\xi$. The results are shown in Eqs 8 and 9.

$$e_\varphi(\xi, \lambda) = a_1(\xi) + a_2(\xi) \times \lambda^2$$  \hspace{1cm} (6)

$$\varphi = \frac{1}{2 \lambda^2} \left[ (a_1 + 1) + (a_2 + 1) \lambda^2 \right] - \sqrt{\left[(a_1 + 1) + (a_2 + 1) \lambda^2 \right]^2 - 4 \lambda^2}$$  \hspace{1cm} (7)

$$a_1 = -0.2\xi^2 + 0.06\xi + 0.10$$  \hspace{1cm} (8)

$$a_2 = 0.09\xi^2 - 0.03\xi + 0.05$$  \hspace{1cm} (9)

**Column curve of Specimens of $\lambda < 1.25$**

Linear functions are chosen as primary function for the least square method, as shown in Eq. 10. Similarly, the results of $b_1$ and $b_2$ are shown in Eqs 11 and 12.

$$\varphi = b_1 \lambda + b_2$$  \hspace{1cm} (10)

$$b_1 = -0.28\xi^2 - 0.54\xi - 0.48$$  \hspace{1cm} (11)

$$b_2 = 0.37\xi^2 + 0.66\xi + 1.13$$  \hspace{1cm} (12)

Taking Eqs 11 and 12 into Eq. 10, $\varphi$ of specimens of $\lambda < 1.25$ can be obtained:

$$\varphi = \min \left\{(-0.28\xi^2 - 0.54\xi - 0.48) \lambda + 0.37\xi^2 + 0.66\xi + 1.13, 1 \right\}$$  \hspace{1cm} (13)

The overall results are compared with the experimental results with relatively good agreement, as shown in Figure 6, where $\varphi_\lambda$ and $\varphi_\text{ex}$ are stability coefficients from column curves and experiment, respectively.
CONCLUSION

Stub and long CFRP-Al hybrid columns exhibited higher compressive bearing capacities and higher buckling loads compared to Al columns, respectively. Specially, for the long one, the loads did not decline suddenly after buckling occurred due to the use of the CFRP jacket. Besides, it was found that the buckling mechanism of long hybrid columns was different from that of pure columns due to the elastic CFRP jacket cannot yield. Thus, based on the Perry-Robertson formulation and the related coefficients determined by the FEM results, column curves of long hybrid columns were proposed, which shows good agreements with experimental results.

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EXPERIMENTAL STUDY ON THE FATIGUE BEHAVIOUR OF CRACKED STEEL BEAMS STRENGTHENED BY DIFFERENT HIGH STRENGTH MATERIALS

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ABSTRACT

Carbon fibre reinforced polymer (CFRP) materials have been proven effective in strengthening steel structures. Extensive studies were conducted to investigate the fatigue behaviour of CFRP retrofitted steel plates while research on repaired steel beams was less reported. This study tested seven cracked beams under cyclic loading, among which, six were strengthened by different materials and patch systems. These materials included normal modulus CFRP laminate, high strength steel (HSS) plate, SafStrip (SAF) plate, and two types of epoxies. The effect of adhesive bonding and mechanical anchorage was discussed. It was found these high strength materials considerably retarded the crack propagation and extended the fatigue life of the test specimens. In some cases, the crack growth rate decreased as the crack length increased. Besides, the crack mouth opening displacement (CMOD) was also significantly reduced after retrofitting. Comparing the fatigue life when the crack propagated to half height of the steel beam, CFRP laminate with structural adhesive Araldite 420 appeared to be the most effective among these retrofitting schemes.

KEYWORDS

Fatigue test, high strength material, steel beam, strengthening.

INTRODUCTION

Fatigue damage is a major concern for many steel structures since fatigue cracks may emanate at areas where stress concentrates. Therefore, it is of great importance to retrofit the deficient members in a cost-effective way. In recent years, externally bonded carbon fibre reinforced polymer (CFRP) was considered as an alternative in comparison with traditional techniques (e.g. Zhao and Zhang 2007; Teng et al. 2012). Research on CFRP application to cracked steel plates has demonstrated the great potential of this approach in fatigue crack repair. Extensive experimental results showed that the overlays could significantly retard crack propagation and extend fatigue life of steel plates (e.g. Täljsten et al. 2009; Liu et al. 2009a; Yu et al. 2013; Colombi et al. 2014) and numerical simulation was performed to investigate the effect of repair schemes, such as bond length, bond width, bond location and number of bond layer on the strengthening efficiency (e.g. Kaddouri et al. 2008; Liu et al. 2009b; Wang et al. 2013; Yu et al. 2014).

However, research work on fatigue strengthening of cracked steel beams is comparatively less performed. Jiao et al. (2012) tested 21 initially cracked steel beams under fatigue loading and three different retrofitting methods were considered, i.e., welding, welding and bonding with CFRP plate or CFRP woven. The experimental results showed that in comparison with solely welding, one layer of CFRP plate and four layers of CFRP woven sheets extended the fatigue life of the steel beams about seven times and three times, respectively. In Wu et al. (2012), the retrofitting efficiency of more strengthening materials was compared, including high-modulus CFRP plate (HM-CFRP), high-strength CFRP plate (HS-CFRP), steel-wire basalt-fibre-reinforced polymer plate (SW-BFRP) and welded steel plate. It was concluded from the current results that the best fatigue performance was achieved by HM-CFRP. Since adhesive is the weakest link in the bonding system of CFRP to steel, the bond behaviour plays an important role which is directly related to the success of the strengthening scheme. Kim and Harries (2011)
presented an experimental and numerical study on the flexural behaviour of damaged steel beams strengthened by CFRP strip. In particular, an empirical model was proposed to estimate the fatigue response of CFRP-steel interface. More recently, Colomi and Fave (2015) also focused on the interfacial behaviour. It was revealed that a debonded area did exist around the crack front. With consideration of the complex issues of adhesive bonding, Ghafouri et al. (2012a,b) developed a prestressed unbonded reinforcement system and derived the required prestressing level to arrest the crack propagation.

In this study, seven defected steel beams strengthened by different high strength materials were tested under fatigue loading, including normal modulus CFRP laminate, high strength steel (HSS) plate and SafStrip (SAF) plate. Both adhesive bonding and mechanical anchorage were adopted based on the properties of these patches. The crack propagation and crack mouth opening displacement (CMOD) with fatigue cycles were recorded during the experimental programme. Test results showed that in comparison with the control specimen, the fatigue behaviour of the steel beams was significantly improved with different high strength materials and repair configurations.

**DESCRIPTION OF TEST SPECIMENS**

**Configuration of Test Specimens**

Specimens were I-shaped steel beams (HM 150 × 75 × 5 × 7) with an overall width of 75 mm and a height of 150 mm. The specimen was machined with a notch to simulate the initial damage. The artificial slot, which was 15 mm long and 0.3 mm wide, went through the tension flange and a portion of the web, as shown in Figure 1.

A total of seven specimens were designed, of which one was tested as the control specimen and the other six were strengthened on the tension flange by using different materials and patch systems (Figure 2). CFRP laminate, HSS plate, and SAF plate all have high strength-to-weight ratio and their nominal thickness were 1.4 mm, 1 mm and 3.2 mm, respectively. Different from conventional CFRP materials, the SAF plate is a combination of unidirectional carbon and E-glass fibers and E-glass continuous strand mats, leading to a high bearing strength. Their mechanical properties are listed in Table 1.

<table>
<thead>
<tr>
<th>Material</th>
<th>Yield strength (MPa)</th>
<th>Ultimate strength (MPa)</th>
<th>Elastic modulus (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>378.2</td>
<td>519.0</td>
<td>192.8</td>
</tr>
<tr>
<td>CFRP (Wu et al. 2015)</td>
<td>N/A</td>
<td>3022.4</td>
<td>200.4</td>
</tr>
<tr>
<td>HSS (Wu et al. 2015)</td>
<td>978</td>
<td>1076</td>
<td>198</td>
</tr>
<tr>
<td>SAF (Wu et al. 2015)</td>
<td>N/A</td>
<td>1071</td>
<td>65.8</td>
</tr>
<tr>
<td>Araldite 420</td>
<td>N/A</td>
<td>N/A</td>
<td>1.495×10^{-3}</td>
</tr>
<tr>
<td>Sikadur 30</td>
<td>N/A</td>
<td>29</td>
<td>11.2×10^{-3}</td>
</tr>
</tbody>
</table>

**Material Properties**

The steel beams used in the tests were Q345b (according to Chinese Standard GB 50017-2003). Coupon specimens were manufactured from the web and flange and the results are listed in Table 1. CFRP laminate, HSS plate and SAF plate all have high strength-to-weight ratio and their nominal thicknesses were 1.4 mm, 1 mm and 3.2 mm, respectively. Different from conventional CFRP materials, the SAF plate is a combination of unidirectional carbon and E-glass fibers and E-glass continuous strand mats, leading to a high bearing strength. Their mechanical
properties were also obtained through coupon tests. Regarding to the cases strengthened by adhesive bonding, two types of epoxies, Araldite 420 and Sikadur 30, were selected to bond the CFRP to the steel substrate. The technical data provided by the manufacturer is given in Table 1.

**Specimen Preparation**

The tension flange was first sandblasted and then cleaned with high-pressure air and acetone. For adhesive bonded specimens, CFRP laminates were glued to the steel surface and the specimens were cured for two weeks at room temperature. The adhesive layer thickness was controlled to be around 0.5 mm. For mechanical anchored specimens, the screws were driven in by a torque wrench with a torque moment of 56 kN·m.

**FATIGUE TESTING**

**Loading Procedure**

Four-point bending tests were performed on all the specimens as shown in Figure 1. Cyclic compressive loading \( P \) from 3.5 to 35 kN with a frequency of 6 Hz was selected. The loading applied by the actuator of the MTS servo hydraulic testing machine was evenly distributed between the two loading points through a spreader beam \( (P/2) \). The maximum load of the spectrum was about 63% of the calculated yield load of the unstrengthened beam, based on the mid-span notched cross-section. In order to prevent out-of-plane displacement of the specimen under cyclic loading, two stoppers were installed on both ends of the bending rig (Figure 3).

Besides fatigue loading, the specimens were also loaded statically to the maximum load of the fatigue loading spectrum at the very beginning and after a certain cycle numbers to detect the CMOD variation. The fatigue test was stopped when the crack broke through the beam web or two million cycles were achieved.

**Data Acquisition**

It is important to monitor the crack propagation with fatigue cycles. Though uncovered, the accurate position of the crack front in the beam web was difficult to identify as a result of the constraint effect of strengthening materials on the crack opening. Therefore, strain gauges at a certain interval were attached to characterize the crack growth. Sudden change of the strain reading indicated that the strain gauge was damaged due to the crack propagation to the corresponding position. During the static test, two clip-shape displacement transducers with a gage length of 2 to 7 mm were installed at both ends of the initial cut on the tension soffit to record the CMOD.

![Fatigue test set-up](Figure 3 Fatigue test set-up)

**Table 2 Experimental programme and results**

<table>
<thead>
<tr>
<th>Beam</th>
<th>Strengthening materials</th>
<th>Patch configuration</th>
<th>Adhesive thickness (mm)</th>
<th>Fatigue life</th>
<th>Failure mode</th>
<th>Fatigue life extension ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam 1</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>16700</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Beam 2</td>
<td>CFRP</td>
<td>Araldite 420</td>
<td>0.57</td>
<td>&gt;2000000</td>
<td>--</td>
<td>N/A</td>
</tr>
<tr>
<td>Beam 3</td>
<td>CFRP</td>
<td>Sikadur 30</td>
<td>0.65</td>
<td>121406</td>
<td>Cohesive failure</td>
<td>7.3</td>
</tr>
<tr>
<td>Beam 4</td>
<td>CFRP</td>
<td>Anchor A</td>
<td>N/A</td>
<td>263259</td>
<td>Anchor failure</td>
<td>15.8</td>
</tr>
<tr>
<td>Beam 5</td>
<td>CFRP</td>
<td>Anchor B</td>
<td>N/A</td>
<td>&gt;2000000</td>
<td>--</td>
<td>N/A</td>
</tr>
<tr>
<td>Beam 6</td>
<td>HSS</td>
<td>Anchor C</td>
<td>N/A</td>
<td>128668</td>
<td>HSS plate fracture</td>
<td>7.7</td>
</tr>
<tr>
<td>Beam 7</td>
<td>SAF</td>
<td>Anchor C</td>
<td>N/A</td>
<td>70198</td>
<td>SAF plate fracture</td>
<td>4.2</td>
</tr>
</tbody>
</table>

**EXPERIMENTAL RESULTS AND DISCUSSIONS**

A total of seven specimens were tested. The detailed experimental programme and the results are listed in Table 2.
Failure Modes

As the cycle number increased, the crack of Beam 1 propagated towards the compression flange and finally went through the beam web (Figure 4). With respect to the specimens with retrofitting, Beams 2 and 5 survived after two million cycles and different failure modes were observed for the other specimens, as depicted in Figure 5.

![Failure of Beam 1 without retrofitting](image1)

Beams 2 and 3 having the CFRP laminates bonded by Araldite 420 and Sikadur 30, respectively. Beam 2 did not fracture after two million cycles and it was found that the crack had already gone throughout the beam web at about 1.07 million cycles. Afterwards, no further crack propagation was observed and the interfacial debonding zone around the initial notch spread towards the ends of the laminate. Similar phenomenon occurred to Beam 3 that interfacial debonding progressed as the fatigue cycle number increased as a result of stress concentration. The debonding progression along the bond length was not exactly symmetrical with respect to the mid-span. When the crack reached a certain length, it suddenly went through the beam web accompanied with cohesive failure.

Beams 4 to 7 were strengthened by mechanical anchorages. In Beams 4 and 5, the CFRP laminates were attached through frictional force. One bolt of Beam 4 fractured, and afterwards, the crack broke through the beam web. Different areas on the fracture surface of the bolt as shown in Figure 5(b) implied that it may fail by fatigue. Beam 5 survived when two million cycles were applied and the crack had not reached 128 mm where a strain gauge was attached, which demonstrated that Anchor B was more effective in retarding crack propagation in comparison with Anchor A. For Beams 6 and 7, which were repaired by HSS and SAF directly anchored to the tensile flange, respectively, the strengthening materials finally fractured. It is found in Figures 5(c) and 5(d) that the HSS plate fractured at the middle cross section whereas the SAF plate fractured at the place of anchor. It was due to the material properties of these two kinds of materials that the HSS plate was a uniform material while the SAF plate was composed of fibres. Though drillable, the holes cut fibres of the SAF plate, resulting in a detrimental effect and consequently a local failure at the anchorage.

![Failure of the steel beams with retrofitting](image2)

Fatigue Crack Propagation and Opening

Figure 6 plots the crack propagation with fatigue cycle numbers of all the specimens. It should be pointed out that, beams 2 and 5 did not fracture during the experimental programme. The crack length was measured from the tension soffit to the crack front, including the dimension of the initial slot. In comparison with the unstrengthened specimen, the crack propagation was significantly retarded in the strengthened specimens. It was noticed that for
Beams 2 and 5 survived after two million cycles, their $N-a$ curves were upper convex, which indicated that the crack propagation rate decreased with crack length.

The average CMOD value recorded by the two displacement transducers versus fatigue cycles is depicted in Figure 7. The results shown here are the values at the peak load of 35 kN during every static loading process. For Beam 1, the vertical slope indicated that the CMOD developed very quickly. With the presence of strengthening materials, the CMOD was considerably decreased, which implied that these overlay patches could offer a constraint effect on the crack opening. Similar to Figure 6, different shapes of CMOD versus cycle number curve were observed that a better performance was achieved in Beams 2 and 5 where the CMOD development was obviously slowed down with fatigue cycles.

![Fatigue Life Crack length versus fatigue cycles](image1)
![Fatigue Life CMOD versus fatigue cycles](image2)

**Fatigue Life**

Table 2 lists the fatigue life and fatigue life extension ratio of all the specimens. The fatigue life extension ratio is defined as the fatigue life of a strengthened specimen to that of the bare specimen. Besides Beams 2 and 5, which didn’t fail at two million cycles, the fatigue life of the strengthened specimens was extended from 4.2 to 15.8 times. Beams 2 and 3 were both retrofitted with adhesive bonding and the test results clearly showed the difference between retrofitting with structural adhesives of Araldite 420 and Sikadur 30. Araldite 420 was more efficient in this strengthening scheme which was mainly attributed to its higher interfacial fracture energy in comparison with Sikadur 30 (Fernando 2010). Beam 4 had its fatigue life prolonged to 15.8 times, which was almost twice that of Beam 3. It was therefore confirmed the flexibility of mechanical anchorage applied in fatigue strengthening. Anchor B showed an apparent superiority to Anchor A, which was because the number of bolts in Anchor B was twice as many as Anchor A, leading to a larger pressure force and consequently frictional force. The fatigue life ratios of Beams 6 and 7 were 7.7 and 4.2 times, respectively, although the nominal tensile stiffness of HSS and SAF was approximately the same. It was mainly due to the detrimental effect of hole drilling to the SAF plate as aforementioned, and as a result, the fatigue life of Beam 7 was relatively shorter.

When the crack reached half height of the beam, the specimen was considered as severely damaged. If we compare the fatigue life at the crack length of 75 mm as shown in Figure 6, it was found that the best performance was achieved by using CFRP laminate with Araldite 420.

**CONCLUSIONS**

This paper presented an experimental study on the fatigue behaviour of defected steel beams strengthened with different high strength materials. The following is concluded.

Based on the experimental investigations, the fatigue life of the strengthened specimens was considerably extended in comparison with the bare steel beam and two specimens survived after two million cycles. The retrofitting technique not only retarded the crack propagation but also reduced the CMOD. In some cases, the development rate of crack length and CMOD decreased with crack length. Considering the fatigue life when the crack reached half height of the specimen, Beam 5 with CFRP bonded by Araldite 420 led to the best performance.

Different failure modes were observed in the experimental programme. The CFRP attached specimens fractured due to cohesive failure or anchor failure while the HSS/SAF attached specimens fractured resulted from the rupture of the strengthening materials. The fatigue behaviour improvement of CFRP retrofitting with mechanical anchorage was comparable to that with adhesive bonding. In comparison with CFRP materials, the strengthening efficiency by using HSS plate and SAF plate was relatively weaker. However, they were thought to be feasible for temporary retrofitting attributed to the ease of installation.
Significant difference in fatigue life was observed for beams retrofitted with Araldite 420 and Sikadur 30. It was concluded based on the current test results that, Araldite 420 was more useful in this strengthening system.

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MODE-I FATIGUE CRACK ARREST IN METALLIC GIRDERS USING PRE-STRESSED UN-BONDED CFRP PLATES

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ABSTRACT

Although there exists many studies that have shown that CFRP-strengthening of cracked steel elements can extend the fatigue life substantially, there is not many researches on the design criterion that can guarantee a complete fatigue crack arrest in steel. This paper aims to give a better understanding about the design of CFRP retrofit solutions that can assure a fatigue crack arrest in damaged steel members subjected to cyclic loads. The concepts of crack closure, fatigue thresholds and effective stress intensity factor (SIF) are briefly explained. A fracture-based analytical solution is used to estimate the overall SIF of a cracked steel beam retrofitted by pre-stressed CFRP plate. In order to verify the accuracy of the proposed retrofit concept, three steel beams were strengthened by pre-stressed un-bonded CFRP plates. Before strengthening, all beams were notched, and subsequently were subjected to pre-cycling to create sharp cracks with different sizes of 3 mm, 20 mm and 40 mm. The specimens were then strengthened with different CFRP pre-stress levels of 10%, 20% and 30%. Finally, the specimens were subjected to cyclic loading. It has been shown that as long as the overall SIF at the crack tip is zero (or negative), the crack does not propagate, and a complete fatigue crack arrest can be achieved.

KEYWORDS

Carbon fibre-reinforced polymer (CFRP), steel strengthening, bridge metallic girders, fatigue crack arrest.

INTRODUCTION

Civil infrastructures are ageing worldwide and require constant inspections and repairs. In some cases the damage is so critical that the whole structures need to be replaced. Especially in the case of bridges, this often requires a huge financial investment and can also lead to traffic congestions for an extended period of time during the demolition of the old structure and construction of a new one. Therefore, there is a need to develop new retrofit techniques and concepts. Carbon fibre-reinforced polymer (CFRP) has been proven to be effective in strengthening of steel plates (e.g., Colombi et al. 2003, Dawood et al. 2007, Schnerch et al. 2006, Teng 2012, Zhao 2013) and steel beams (e.g., Ghafoori 2013, Ghafoori and Motavalli 2013, Ghafoori and Motavalli 2015b, Ghafoori and Motavalli 2015c, Ghafoori and Motavalli 2015a).

In general, fatigue strengthening of steel members can be done for members with or without cracks. In the latter case, strengthening is done to prevent fatigue crack initiation (e.g., Ghafoori et al. 2015a, Ghafoori et al. 2015b, Ghafoori et al. 2015c, Ghafoori et al. 2014), and in the former case, to reduce or completely arrest fatigue crack growth (FCG) of an existing crack (e.g., Aljabar et al. 2016, Ghafoori and Motavalli 2011, Ghafoori et al. 2012a, Ghafoori et al. 2012b). This paper focuses on the former case (i.e., strengthening of metallic elements with existing crack). Although many studies have shown that CFRP strengthening can extend the fatigue life of the steel elements considerably (e.g., Colombi et al. 2014, Colombi et al. 2015, Dawood et al. 2007, Wu et al. 2012, Wu et al. 2013), there are not many studies on the conditions of a complete arrest of FCG. Therefore, this study aims to give a better understanding about the criterion that results in a complete fatigue crack arrest in cracked steel members.

BASICS OF FATIGUE AND FRACTURE MECHANICS

As this paper aims to present a CFRP strengthening approach for fatigue crack arrest in cracked steel members, there is a need to have a basic knowledge about some key concepts in fracture mechanics such as crack closure phenomena and fatigue threshold, which are briefly explained in this section.
Crack Closure Effect (CCE)

The crack closure concept implies that the elastic compliance of cracked specimen at low loads remains identical to the compliance of un-cracked specimen because the crack faces are still in contact. The most common five mechanisms that lead to crack closure effect (CCE) are plasticity-induced, roughness-induced, oxide-induced, induced by viscous fluids and transformation-induced crack closure. The CCE results in a certain tensile load before the crack starts to open (K_{op}). In this paper, K is the stress intensity factor (SIF). The effective SIF range is then defined as (Elber 1970)

\[ \Delta K_{eff} = K_{max} - K_{op} \]  

which results in the modified Paris law

\[ \frac{da}{dN} = C \cdot \Delta K_{eff}^m \]  

where a and N are the crack length and number of cycles, respectively, and C and m are material constants. This phenomenon is illustrated in Figures 1.a and 1.b. The crack opening SIF, K_{op}, can either be determined directly by measuring the displacement at the crack mouth or from the compliance of the whole specimen. In that case, K_{op} (see Figure 1.a) is defined at the intersection between the un-cracked and cracked stiffness (see Figure 1.b). Nevertheless, the transition from fully closed to fully open happens over a certain load range, as shown in Figure 1.b. Depending on the crack size, it is also possible that the crack tip remains open while the crack faces are partially closed. For a partially open crack with a fully closed crack tip as well as for a fully closed crack, it can be assumed that the SIF is zero (Ghafoori et al. 2012a, Beghini and Bertini 1990).

Fatigue Threshold Concept

The ASTM Standard E647−13a 2014 defines the fatigue threshold as the ΔK-value linearly extrapolated for fatigue growth rate of 10^{-10} m/cycles. The crack propagation rate in steel is depended on the stress ratio (R) (see Figure 2), which is generally believed to be due to the CCE. The effect disappears when the values for low R ratios are corrected for ΔK_{eff} and in that case all data converges to the same value of ΔK_{th}, as can be seen in Figure 2 (Tanaka 1989).

For the case of a CFRP-strengthened steel beam, as shown in Figure 3, the crack remains closed due to the compression from the pre-stressed CFRP plate up to a certain amount of the external load. Only the load range between the crack opening load and the upper load limit is effective. Therefore, the concept of ΔK_{eff} is used in this paper to calculate the SIF of the cracked members.

FRACTURE ANALYSIS OF CFRP-STRENGTHENED STEEL BEAMS

Crack Closure Phenomena Triggered by Pre-stressed CFRP Plate

From Figure 3, the pre-stressed CFRP plate that is attached to the beam bottom flange applies a compressive force to the cracked section. The pre-stressing load also results in an additional bending moment due to the eccentricity of the CFRP plate from the beam neutral axis. Hence, the acting forces on the steel beam have three components: I) a bending moment due to the external load, II) a compressive axial force due to the pre-stressed CFRP plate, and, III) a negative bending moment due to the eccentricity of the CFRP plate. The fracture effect of these three force components on the crack in the steel beam can be summed up using superposition rule, which is valid in the linear elastic fracture mechanics (LEFM). Note that the force components II and III, due using the CFRP plate, are acting against the force component I. Depending on the magnitude of the external load and the CFRP pre-stressing force, the stress distribution at the cracked-section changes, as shown in Table 1. In the absence of the external bending moment, the existing crack is completely under compression due to the compressive stress from the pre-
stressed CFRP plate. With a slight increase of the external load, the compressive stress at the crack is reduced until the crack-opening moment \( M_{co} \) is reached, implying that the very bottom part of the crack is stress free. If the external load is further increased, the crack will be in the semi-active state which means that part of the crack is under compression. The crack becomes active only when that the crack tip is no longer under compression, and, therefore, the crack-activation moment \( M_{ca} \) is reached. This also represents the instant when the mode-I stress intensity factor \( K_I \) is exactly zero. For external bending moments larger than \( M_{ca} \), stresses at the crack tip increase and the resultant \( K_I \) becomes larger.

Figure 2 \( \Delta K_{th} \) for mild steel (Tanaka 1989).

Note that in some literature and especially in many numerical simulations, a crack under compression is related to a negative stress intensity factor. Although a negative value for the SIF does not have a physical meaning, it implies that the crack tip is under compression (e.g., Ghafoori et al. 2012a, Ghafoori et al. 2012b).

Table 1 Strain distribution along the cracked section based on the magnitude of the external load and the CFRP pre-stressing level.

<table>
<thead>
<tr>
<th>Crack mode</th>
<th>passive</th>
<th>Semi-active</th>
<th>active</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strain distribution</td>
<td><img src="image1.png" alt="Strain分布图" /></td>
<td><img src="image2.png" alt="Strain分布图" /></td>
<td><img src="image3.png" alt="Strain分布图" /></td>
</tr>
<tr>
<td>Applied moment</td>
<td>( M = 0 )</td>
<td>( M = M_{co} )</td>
<td>( M_{co} &lt; M &lt; M_{ca} )</td>
</tr>
<tr>
<td>Stress intensity factor</td>
<td>whole crack under compression</td>
<td>whole crack under compression</td>
<td>crack tip under compression</td>
</tr>
</tbody>
</table>

**SIF of Steel I-Beams Strengthened with Pre-stressed CFRP Plates**

An analytical model for the SIF of cracked I-beams under axial load \( (N) \) and bending moment \( (M) \) has been developed based on the energy release rate by Ghafoori and Motavalli 2011

\[
K_I = \left[ -\frac{N^2}{A} \cdot \frac{(M - Y_c N)^2}{I} + N^2 \cdot (\lambda_1 + \lambda_2) + M^2 \cdot (\eta_1 + \eta_2) \right] \frac{\pi}{t_{ef}(1 - \nu^2)} \]  

(3)

where \( Y_c \) is the position of the neutral axis of the cracked and un-cracked cross section. \( \lambda_i \) and \( \eta_i \) are parameters based on the geometry of the cross-section and the crack length. The subscript \( c \) denotes to the cracked cross section. Ghafoori et al. 2012a have extended their analytical method for calculation of the overall SIF of the CFRP-
strengthened beams by superpositioning the SIFs related to the different load components. Assuming that the LEFM conditions are fulfilled, the overall SIF of the CFRP-strengthened beams is given by

$$K_{overall} = K_{external\ load} + K_{CFRP} = K_1^{external\ load} + K_1^{max} + K_2^{max}$$  \hspace{1cm} (4)

Hence, the overall SIF as a function of the geometry, the external bending moment and the pre-stressing level is given by (Ghafoori et al. 2012a)

$$K_1^{overall} = \frac{M - P_l(t_f + \frac{h}{2})}{\pi t_w(1 - \nu^2)} \left[ \left( \frac{1}{l} + (\eta_1 + \eta_2) \right) \frac{\pi}{t_w(1 - \nu^2)} \right]^{\frac{1}{2}} - \frac{P_l}{A} \frac{(1 + \lambda_1 + \lambda_2) + Y_e(\eta_1 + \eta_2)}{t_w(1 - \nu^2)}$$

where $P_l$ is the current axial force in the CFRP under the external bending moment $M$. More details about the analytical solution can be found in Ghafoori et al. 2012a.

**EXPERIMENTAL PROGRAM**

Materials and Specimens

The CFRP plates were provided by the S&P Clever Reinforcement Company AG, Switzerland. The plates had a width of 50 mm and a thickness of 1.2 mm. The mechanical properties of the CFRP, based on the production data sheet, are listed in Table 2. In this paper, all pre-stressing levels are expressed as a percentage of the tensile strength of the CFRP ($f_{t, CFRP} = 2704$ MPa). The IPE 120 steel beams were provided by Briner AG, Switzerland. The mechanical properties of the steel profiles, based on the production data sheet, are listed in Table 3. Three steel beams were strengthened using unbonded CFRP plates. The procedure of strengthening is similar to what has been explained by Ghafoori et al. 2012a. The tests on the beams were performed using the four-point bending test set-up, shown in Figure 3.

<table>
<thead>
<tr>
<th>Table 2 Mechanical properties of the CFRP plates.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Product name</strong></td>
</tr>
<tr>
<td>S&amp;P150/2000 50/1.2</td>
</tr>
</tbody>
</table>

In order to create a starting point for the crack to grow, an initial damage in the form of a notch was created in the mid-section of the beam. Therefore, the bottom flange of each beam was cut at the mid-span and a 1.5 mm wide and 6 mm long notch was cut into the web, as can be seen in Figure 4. The geometry of notch size was identical for all specimens. Each specimen, however, was subjected to a pre-cycling loading regime so that the crack length is increased to a desired length according to ASTM Standard E647–13a 2014. Details about pre-cycling will be given later in this paper. Finally, three beams with the crack lengths of 3 mm, 20 mm and 40 mm were prepared, as shown in Table 4. Each specimen has been retrofitted by one CFRP plate with a particular pre-stress level, as indicated in Table 4. In this paper, due to the limitation of paper size, only details about preparation and test results of specimen Beam 1 is provided and more details about the whole experimental, numerical and analytical research program will be published later in a journal paper.

<table>
<thead>
<tr>
<th>Table 3 Mechanical properties of the S355 steel according to EN 10025.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Steel grade</strong></td>
</tr>
<tr>
<td>S355JR</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 4 Test specimens and loadings.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Specimen</strong></td>
</tr>
<tr>
<td>Beam 1</td>
</tr>
<tr>
<td>Beam 2</td>
</tr>
<tr>
<td>Beam 3</td>
</tr>
</tbody>
</table>

**Pre-cycling**

The crack in all three specimens was grown by applying pre-cyclic loading according to ASTM Standard E647–13a 2014, as summarized in Table 4. According to the ASTM Standard, the crack propagation rate was kept below $10^{-3}$ m/cycles. The size of the plastic zone was estimated to be small enough to guarantee that the condition for LEFM is fulfilled. The length of fatigue crack after pre-cycling for Beam 1, Beam 2 and Beam 3 were 3 mm, 20 mm and 40 mm, respectively. A strain gauge was glued near to the crack tip (see Figures 4 and 5) to estimate the stress level near the crack tip. After applying the pre-cycling, the specimens were strengthened with the pre-stressed CFRP plates (see Table 4).
Figure 4 A strain gauge attached at the vicinity of the crack tip in a) Beam 1 and b) Beam 3.

Figure 5 Strain at the crack tip for Beam 1 during cyclic loading as function of a) external load, and b) time.

Figure 6 Visual inspection of the crack before and after the fatigue loading as well as after the increased load.

**Cyclic Loading**

For strengthening Beam 1, the CFRP was first pre-stressed up to 264.9 MPa (9.8% of the failure stress) before the beam is placed on the CFRP and the clamps are tightened. After the release of the actuator load, the stress in the CFRP is reduced to 239.1 MPa (i.e., about 10% decrease). For fatigue test, the upper and the lower loads were set...
to 6.0 kN and 0.6 kN, respectively. This results in a stress ratio of $R=0.1$. The upper load level was chosen in a way that results in a zero strain at the crack tip (i.e., onset of crack activation, $M_0$). The magnitude of the stress at the crack tip was read by the applied strain gauge, as shown in Figure 4. 1’000’000 fatigue cycles with a loading frequency of 4.35 Hz were applied. Figures 5.a and 5.b show the strain at the crack tip during cyclic loading as function of force in cylinder and time, respectively.

No fatigue crack growth was observed after 1’000’000 cycles. This observation acknowledges that fact that the crack does not grow as long as the crack tip is not in active mode (see Table 1). Note that the SIF of the retrofitted beam according to Eq. (5) is zero. Furthermore, Figures 6.a show the visual inspection of the crack after pre-cycling, while Figure 6.b depicts the state of crack after strengthening and application of 1’000’000 cycles, which indicates a complete fatigue crack arrest.

### Increased Cyclic Load Level

After 1’000’000 cyclic loads without any crack propagation in the steel, the load was increased just above the fatigue threshold for additional 200’000 cycles. The threshold value of SIF (i.e., $\Delta K_{th}$) for mild steel based on Figure 2 is about 2.8 MPa$\sqrt{m}$, which is equal to 88.3 MPa$\sqrt{mm}$. As the strengthening with the pre-stressed CFRP results in a crack closure due to the compressive force, it is appropriate to use the data points for $\Delta K_{eff}$ in Figure 2. The increased load level was aimed to create a FCG rate of $10^{-5}$ mm/cycle (i.e., a crack growth of 2 mm in 200’000 cycles). As it is shown in Figure 2 by two dashed red lines, this FCG rate corresponds to a $\Delta K_{eff}$ of 9 MPa$\sqrt{m}$, which is equivalent to 284.6 MPa$\sqrt{mm}$. The analytical formulation in Eq. (5) was used to calculate the load corresponding to $\Delta K_{eff}=284.6$ MPa$\sqrt{mm}$, which resulted in an upper and lower load levels of 7.8 kN and 0.8 kN. The beam was then subjected to this cyclic load range and the crack propagated about 1.7 mm with an average crack propagation rate of $8.5 \times 10^{-6}$ mm/cycle. Figures 7.a and 7.b show the strain at the crack tip for Beam 1 after increased load level as a function of external load and time, respectively. It can be seen that the crack tip is under tension, which implies an active crack mode. Furthermore, Figure 6.c clearly shows that the crack length was increased for about 1.7 mm after applying 200’000 cycles with increased load.

![Figure 7](image-url)

Figure 7. Strain at the crack tip for Beam 1 after increased load level as function of a) external load, and b) time.

Similar test procedure has been repeated for Beam 2 with medium crack length and Beam 3 with long crack length. The results of the other two tests were similar to the results of Beam 1.

### CONCLUSIONS

Basic concepts of fracture mechanics such as the crack closure effect, the fatigue threshold, and the effective SIF have been briefly explained. For the case of the CFRP-strengthened cracked steel elements, different fracture modes of passive, semi-active and active crack modes have been explained. It has been shown that the CFRP pre-stress can act as a source for the crack closure.

Three steel beams were notched and then subjected to pre-cycle loading regime to create sharp cracks from the notches with different sizes of 3 mm, 20 mm and 40 mm. The specimens were strengthened with different CFRP pre-stress levels of 10%, 20% and 30%. The specimens were then subjected to cyclic loading that would result in a zero SIF at the crack tip (i.e., onset of crack activation). It has been shown that once the overall SIF at the crack tip is zero (or negative), the crack does propagate. In other words, when the CFRP pre-stress level is high enough.
to apply a compressive stress to the crack tip, the crack remains passive and the FCG is arrested. In order to further verify this conclusion, the cyclic loading level were slightly increased such that the resulting SIF became positive (i.e., crack tip under tension). The latter resulted in an active crack mode, therefore, fatigue crack propagated though the web. The main finding of this research is to show that it is possible to design a CFRP retrofit solution for cracked steel members such that the fatigue crack propagation is arrested.

REFERENCES
Journal of Constructional Steel Research, 78, 131-143.


COMPARISON AND ANALYSIS OF THE EFFECT OF STRENGTHENING STEEL PLATE WITH VOLUME TYPE DEFECTS BY CFRP ON ONE SIDE

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Southwest University of Science and Technology, Mianyang 621010, China

ABSTRACT

CFRP strengthening effect of steel structure is affected by defect types, pasting location of CFRP and other factors. It is analyzed that the effect regularity of defect size and depth of the defects to bearing capacity of CFRP reinforcing steel, and the influence of relationship between CFRP and defect position on strengthening effect in this paper based on the tensile test of the volume defects of steel plate specimens strengthened by CFRP.

KEYWORDS:
Reinforcement in defect surface, reinforcement in non defect surface, reinforcement effect, defect types.

INTRODUCTION

Steel structure is widely used in civil engineering field because of its high strength, light weight, high rigidity and so on. However, the metal material is easy to be destroyed by chemical reaction with the surrounding environment. But, the corrosion of steel is not uniform. This is easy to cause surface corrosion of the steel structures and stress concentration to affect the safety of steel structure[2-5]. Technology of steel structure strengthened with FRP has the advantages of no secondary stress and defects, good durability, convenient construction, low maintenance costs and has attracted much attention in the field of Engineering[6-13]. However, the effects of FRP strengthening the steel structure with defects are affected by many factors. Hongbo Liu[14] proved that the width, length and thickness of CFRP affected the bearing capacity of steel plate by testing. Ma Jianxun et al.[15] found that the larger bonding area of CFRP is, the more the residual bearing capacity of the steel plate is increased. Lu Yiyun[16] discussed the effects of CFRP bonding width, length and number of layers on the bearing capacity of the specimens.

For FRP strengthening steel structure with defects, we can directly paste the FRP in the surface of defect to complete the reinforcement. If you can not or inconvenience in the defect surface to complete the FRP paste, then you can paste it in the back side of the surface of defect. However, the reinforcement effect of the two methods remains to be seen whether the reinforcement effect is consistent or not. This paper makes a further validation analysis.

EXPERIMENTAL DESIGN

This test specimen is 5mm thick Q235 steel plate. CFRP cloth is UT70-30 unidirectional carbon fiber cloth produced by Sichuan aerospace Tuoxin basalt Industrial Co.Ltd.. Mechanical properties are shown in Table 1. Resin binder is CH-1A impregnated adhesive produced by Longchang Sichuan Chengu Rubber Industry Co. Ltd..

<table>
<thead>
<tr>
<th>Model</th>
<th>Elastic Modulus /10^3MPa</th>
<th>Yield Strength /MPa</th>
<th>Ultimate Strength /MPa</th>
<th>Ultimate Strain</th>
<th>Elongation/%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q235</td>
<td>2.15</td>
<td>310</td>
<td>470</td>
<td>1542</td>
<td>37.9</td>
</tr>
<tr>
<td>UT70-30</td>
<td>2.18</td>
<td>—</td>
<td>3788</td>
<td>—</td>
<td>1.7</td>
</tr>
</tbody>
</table>

The shape of steel corrosion defects is much more variable and irregular. According to the size of the defect area, it is divided into two types: large area corrosion and small area of pitting corrosion. As shown in Figure 1 and Figure 2, the test set with large area defects of steel specimens and small area defects of steel specimens. Among
them, the steel plate specimens with large area defect are non-penetrating oval pit defects, the maximum defect depth is 2mm, and the corrosion area $A_0$ is the percentage of steel plate area. The steel plate specimens with small area defect are non-penetrating conical pit defects which has 10mm long axis and 8mm short axis. The design condition of the specimens is shown in Table 2 and Table 3.

![Figure 1: The Steel Plate Specimens with Large Area Defect](image)

![Figure 1: The Steel Plate Specimens with Small Area Defect](image)

**Table 2** The Design Condition of the Specimens with Large Area Defect

<table>
<thead>
<tr>
<th>Number</th>
<th>Type</th>
<th>Defect area $A_0$/%</th>
<th>Reinforcement method</th>
<th>Long axis of surface defect $2a$/mm</th>
<th>Short axis of surface defect $2b$/mm</th>
<th>Layers of CFRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-20%-1</td>
<td>I-X%-1</td>
<td>20</td>
<td>Non-reinforced</td>
<td>78</td>
<td>36</td>
<td>—</td>
</tr>
<tr>
<td>I-40%-1</td>
<td>I-X%-1</td>
<td>40</td>
<td>Non-reinforced</td>
<td>112</td>
<td>50</td>
<td>—</td>
</tr>
<tr>
<td>I-60%-1</td>
<td>I-X%-1</td>
<td>60</td>
<td>Non-reinforced</td>
<td>136</td>
<td>60</td>
<td>—</td>
</tr>
<tr>
<td>I-20%-2</td>
<td>I-X%-2</td>
<td>20</td>
<td>On the back</td>
<td>78</td>
<td>36</td>
<td>One layer</td>
</tr>
<tr>
<td>I-40%-2</td>
<td>I-X%-2</td>
<td>40</td>
<td>On the back</td>
<td>112</td>
<td>50</td>
<td>One layer</td>
</tr>
<tr>
<td>I-60%-2</td>
<td>I-X%-2</td>
<td>60</td>
<td>On the back</td>
<td>136</td>
<td>60</td>
<td>One layer</td>
</tr>
<tr>
<td>I-20%-3</td>
<td>I-X%-3</td>
<td>20</td>
<td>In the front</td>
<td>78</td>
<td>36</td>
<td>One layer</td>
</tr>
<tr>
<td>I-40%-3</td>
<td>I-X%-3</td>
<td>40</td>
<td>In the front</td>
<td>112</td>
<td>50</td>
<td>One layer</td>
</tr>
<tr>
<td>I-60%-3</td>
<td>I-X%-3</td>
<td>60</td>
<td>In the front</td>
<td>136</td>
<td>60</td>
<td>One layer</td>
</tr>
</tbody>
</table>

**Table 3** The Design Condition of the Specimens with Small Area Defect

<table>
<thead>
<tr>
<th>Number</th>
<th>Type</th>
<th>Maximum defect depth $h$/mm</th>
<th>Reinforcement method</th>
<th>Pasting length /mm</th>
<th>Pasting width /mm</th>
<th>Layers of CFRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>II-1.5-1</td>
<td>II-X-1</td>
<td>1.5</td>
<td>Non-reinforced</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>II-2.5-1</td>
<td>II-X-1</td>
<td>2.5</td>
<td>Non-reinforced</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>II-3.5-1</td>
<td>II-X-1</td>
<td>3.5</td>
<td>Non-reinforced</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>II-1.5-2</td>
<td>II-X-2</td>
<td>1.5</td>
<td>On the back</td>
<td>180</td>
<td>60</td>
<td>One layer</td>
</tr>
<tr>
<td>II-2.5-2</td>
<td>II-X-2</td>
<td>2.5</td>
<td>On the back</td>
<td>180</td>
<td>60</td>
<td>One layer</td>
</tr>
<tr>
<td>II-3.5-2</td>
<td>II-X-2</td>
<td>3.5</td>
<td>On the back</td>
<td>180</td>
<td>60</td>
<td>One layer</td>
</tr>
<tr>
<td>II-1.5-3</td>
<td>II-X-3</td>
<td>1.5</td>
<td>In the front</td>
<td>180</td>
<td>60</td>
<td>One layer</td>
</tr>
<tr>
<td>II-2.5-3</td>
<td>II-X-3</td>
<td>2.5</td>
<td>In the front</td>
<td>180</td>
<td>60</td>
<td>One layer</td>
</tr>
<tr>
<td>II-3.5-3</td>
<td>II-X-3</td>
<td>3.5</td>
<td>In the front</td>
<td>180</td>
<td>60</td>
<td>One layer</td>
</tr>
</tbody>
</table>
THE LOADING PROCESS AND PHENOMENON

Using 300kN microcomputer control electron universal testing machines (max test force 300kN) to carry out tensile test which loading rate is 0.1kN/s. The experiments are carried out in order to test the tensile test of each specimen.

The phenomenon occurred in the whole test process of the large area defect specimen after the front reinforcement and the back reinforcement: The CFRP cloth and the steel plate did not stripping in the initial stage of loading. When the tensile load is close to the yield load of the non-reinforced steel plate, the tensile test piece begin to give off a faint sound. When the tensile load is more than 85kN, CFRP cloth first stripped in the middle of the paste. Along with the increase of the load, the peeling is gradually developed to the end anchorage zone. When the load is close to 90kN, one side of the anchorage began to slip. The deformation of steel plate increases quickly when the end anchorage has a large slip. The middle part of the plate appears necking, then it quickly tensile failure from the defect of the deepest necking position. The angle between the fracture direction and the axis is less than 45 degrees. In this process, CFRP cloth has not been torn or broken. The failure process of the specimen is shown in Figure 3.

The phenomenon occurred in the whole test process of the small area defect specimen after the front reinforcement and the back reinforcement: In addition to the next few points, it is consistent with the loading process of large area corrosion defect specimens. When the tensile load is close to 95kN, the tensile test piece began to give off a faint sound. When the tensile load is more than 110kN, CFRP cloth first stripped in the middle of the paste. The cracking deformation of steel plate is obvious. The angle between the fracture direction and the axis is less than 90 degrees. In this process, CFRP cloth has not been torn or broken. The failure process of the specimen is shown in Figure 4.

Analysis of experimental phenomena: Prior to the yield of the steel plate, the deformation of the CFRP cloth and the steel plate was almost the same, and there was no relative slip, so it can work together well. CFRP cloth share a part of the load, the yield of the steel plate is delayed, and the yield bearing capacity of the member is improved. The end anchoring of CFRP cloth can improve the bonding ability of the end, and delayed stripping of the end of the CFRP cloth, and the tensile load of the component is further improved. After steel plate yield, because the steel plate in the yield stage will have a larger elongation, and the elongation of CFRP cloth still keeps the linear growth. At the same time, the shear strength of the bonding layer can not contentment the bond, so they began to appear relative slip. They can not work together. The plate is tensile failure, because the load quickly transferred to the steel plate. Due to the tensile strength of CFRP cloth is much higher than that of steel, so the CFRP cloth is not broken. When the load continues to increase, the end of the CFRP cloth appears to slip, so it can not further improve the ultimate bearing capacity of the specimen. Because the CFRP cloth is not yielding stage, the test curve showing the strength performance of CFRP cloth. As a result, there is no obvious phenomenon in the yield stage when loading the specimen. The end anchoring of CFRP cloth can improve the bonding ability of the end, and the end of stripping is delayed, and the tensile load of the component is further improved. The test finally end up as the steel plate tensile failure after bonding failure of CFRP cloth and steel plate. If we can increase the bonding strength is a great help to improve the bearing capacity of the specimen.
EXPERIMENT RESULTS AND ANALYSIS

Strengthening specimen with large area defect by CFRP

Load-displacement curve

The load-displacement curve of different defect area is shown in Figure 5. Figure 5 shows that with the increase of the corrosion area, the bearing capacity of all specimens are reduced. Because the maximum defect depth of the test plate is consistent, with the increase of the defect area, the effective section of the steel plate becomes smaller, so the residual capacity is reduced.

![Figure 5](image1)

**Figure 5** The load-displacement curve of specimens with large area defect

Comparative analysis

In Figure 6, the comparative of the load-displacement curves of the corrosion area of 20%, 40% and 60% are given respectively. From the figure shows that the bearing capacity of the reinforcement specimens are all increased. The degree of improvement of yield load of the two types of reinforcement is higher than that of the ultimate load. The yield load mostly increased by 33.2%, The ultimate load mostly increased by 2.55%. For large area defect steel plate, reinforced in the defect surface or non defect surface both can achieve good results. Prior to the yield of the steel plate, the deformation of the CFRP cloth and the steel plate was almost the same, and there was no relative slip, so it can work together well. CFRP cloth share a part of the load, the yield of the steel plate is delayed, and the yield bearing capacity of the member is improved. When the steel plate to the yield stage, the deformation of the steel plate was significantly greater than the deformation CFRP cloth, then they began to appear relative slip, and CFRP cloth is in a state of detachment. The load that CFRP cloth shared quickly transferred to the steel plate. Lead the steel plate enters the yield stage, and its strain increases rapidly. But the deformation of CFRP cloth is very small. So they can not work together well when the steel plate is subjected to a continuous increase of load. Under the effect of super large load, The connection force between steel plate and CFRP cloth is only provided by the end anchorage. So the stripping is also rapidly from the middle to both ends. Because CFRP cloth is slip in the end anchorage zone, the steel plate lost constraint. After that, the deformation of plate increases quickly. Therefore, the ultimate bearing capacity of steel plate is not much improved.

![Figure 6](image2)

**Figure 6** The load-displacement curve of the specimens with large defects which reinforcement and non-reinforcement

The test data of the specimens with large area corrosion defects are remitted to Table 4. Table 4 data show that, compared with the specimens that reinforced on the back, under different corrosion area, the yield load of the specimens that reinforced in the front is respectively increased by 0.95%, 2.17%, 2.92%. The ultimate load is respectively increased by 0.84%, 0.83% and 0.90%. The yield load is increased by approximately 1%-3%. The ultimate load is increased by about 1%. For large area defect of the specimen, the effect of reinforcement in the front is better than...
the effect of reinforcement on the back. Because when the CFRP cloth is directly stuck on the defect surface, the confinement effect of the steel plate is more obvious. This can effectively reduce the stress concentration at the steel plate defects. When the CFRP cloth is far away from the defect surface, the limiting effect is weakened. Therefore, the improvement of the bearing capacity of the composite member is relatively low. But the enhancing effect is very small. So in the future reinforcement, we should firstly consider strengthening in defect surface. If the reinforcement is not achieved in the defect surface, the effect of reinforcement on the back is also very outstanding.

Table 4 Test Result

<table>
<thead>
<tr>
<th>Number</th>
<th>Defect area A0/%</th>
<th>Yield load /kN</th>
<th>Ultimate load /kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Test data /kN</td>
<td>Improvement /%</td>
</tr>
<tr>
<td>I-20%-1</td>
<td>20</td>
<td>80.4</td>
<td>---</td>
</tr>
<tr>
<td>I-20%-2</td>
<td>40</td>
<td>94.5</td>
<td>17.5</td>
</tr>
<tr>
<td>I-20%-3</td>
<td>60</td>
<td>95.4</td>
<td>18.7</td>
</tr>
<tr>
<td>I-40%-1</td>
<td>20</td>
<td>71.0</td>
<td>---</td>
</tr>
<tr>
<td>I-40%-2</td>
<td>40</td>
<td>87.7</td>
<td>23.5</td>
</tr>
<tr>
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<td>26.2</td>
</tr>
<tr>
<td>I-60%-1</td>
<td>20</td>
<td>63.6</td>
<td>---</td>
</tr>
<tr>
<td>I-60%-2</td>
<td>40</td>
<td>82.3</td>
<td>29.4</td>
</tr>
<tr>
<td>I-60%-3</td>
<td>60</td>
<td>84.7</td>
<td>33.2</td>
</tr>
</tbody>
</table>

From the test data in Table 4, When the corrosion area is 20%, 40% and 60%, the yield load of the specimens which reinforced on the back is respectively increased by 17.5%, 23.5% and 29.4%. The yield load of the specimens which reinforced in the front is respectively increased by 18.7%, 26.2% and 33.2%. The larger the corrosion area, the better the reinforcement effect. Because when the corrosion depth is certain, the defect area is bigger, the stress concentration is more relaxed, so the strengthening effect is more obvious.

Strengthening specimen with small area defect by CFRP

Load-displacement curve

The load-displacement curve of different defect depth is shown in Figure 7. From the Figure 7 shows that with the increase of the corrosion depth, the bearing capacity of all specimens are reduced. Because the maximum defect area of the test plate is consistent, with the increase of the defect depth, the effective section of the steel plate becomes smaller, so the residual capacity is reduced.

Comparative analysis

In Figure 8, the comparative of the load-displacement curves of the corrosion depth of 1.5mm, 2.5mm and 3.5mm are given respectively. From the figure shows that the bearing capacity of the reinforcement specimens are all increased. But from the whole experiment, the increase of yield load and ultimate load is not large. The yield load mostly increased by 7.1%, The ultimate load mostly increased by 8.1%. Prior to the yield of the steel plate, the deformation of the CFRP cloth and the steel plate was almost the same, and there was no relative slip, so it can work together well. CFRP cloth share a part of the load. However, although the tensile strength of CFRP is very high, the bond strength of the resin adhesive is very low. So the interface between the steel plate and the resin adhesive is still the weak part of the whole specimen. In addition, from the size of the defect, the steel plate specimens are relatively complete. Therefore, it is impossible to greatly improve the bearing capacity of the specimens under the condition that the shear strength of the resin adhesive is not high.
The test data of the specimens with large small corrosion defects are remitted to Table 5. Table 4 data show that, compared with the specimens that reinforced on the back, under different corrosion depth, the yield load of the specimens that reinforced in the front is respectively increased by 2.37%, 1.70%, 3.50%. The ultimate load is respectively increased by 1.40%, 0.70% and 0.28%. For small area defect of the specimen, the effect of reinforcement in the front is also better than the effect of reinforcement on the back. On the whole, this is not particularly significant to improve the bearing capacity of the bearing capacity of specimens by paste a layer of CFRP cloth. At the same time, it also shows that the relative position of the carbon fiber cloth and the defect is directly related to the improvement of bearing capacity. Because when the CFRP cloth is directly stuck on the defect surface, the confinement effect of the steel plate is more obvious. This can effectively reduce the stress concentration at the steel plate defects. When the CFRP cloth is far away from the defect surface, the limiting effect is weakened. Therefore, the improvement of the bearing capacity of the composite member is relatively low.

![Figure 8](image)

**Figure 8** The load-displacement curve of the specimens with small defects which reinforcement and non-reinforcement

<table>
<thead>
<tr>
<th>Number</th>
<th>Defect depth h/mm</th>
<th>Yield load /kN</th>
<th>Improvement /%</th>
<th>Ultimate load /kN</th>
<th>Improvement /%</th>
</tr>
</thead>
<tbody>
<tr>
<td>II-1.5-1</td>
<td>1.5</td>
<td>97.6</td>
<td>—</td>
<td>—</td>
<td>138.4</td>
</tr>
<tr>
<td>II-1.5-2</td>
<td>1.5</td>
<td>101.2</td>
<td>3.7</td>
<td>—</td>
<td>142.6</td>
</tr>
<tr>
<td>II-1.5-3</td>
<td>1.5</td>
<td>103.6</td>
<td>6.1</td>
<td>2.37</td>
<td>144.6</td>
</tr>
<tr>
<td>II-2.5-1</td>
<td>2.5</td>
<td>95.5</td>
<td>—</td>
<td>—</td>
<td>133.4</td>
</tr>
<tr>
<td>II-2.5-2</td>
<td>2.5</td>
<td>100.1</td>
<td>4.8</td>
<td>—</td>
<td>142.3</td>
</tr>
<tr>
<td>II-2.5-3</td>
<td>2.5</td>
<td>101.8</td>
<td>6.6</td>
<td>1.70</td>
<td>143.3</td>
</tr>
<tr>
<td>II-3.5-1</td>
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<td>93.8</td>
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<td>II-3.5-2</td>
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<td>97.1</td>
<td>3.5</td>
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<td>141.5</td>
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<tr>
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<td>3.5</td>
<td>100.5</td>
<td>7.1</td>
<td>3.50</td>
<td>141.9</td>
</tr>
</tbody>
</table>

From the test data in Table 5. When the corrosion depth is 1.5mm, 2.5mm and 3.5mm, the yield load of the specimens which reinforced on the back is respectively increased by 3.7%, 4.8% and 3.5%. Their ultimate load is respectively increased by 3.0%, 6.7% and 7.8%. The yield load of the specimens which reinforced in the front is respectively increased by 6.1%, 6.6% and 7.1%. Their ultimate load is respectively increased by 4.5%, 7.4% and 8.1%. The deeper the corrosion area, the better the reinforcement effect. Because when the corrosion area is certain, the defect depth is bigger, the stress concentration is more relaxed, so the strengthening effect is more obvious.

**CONCLUSIONS**

1. Whether it is a large area of defect specimen or a small-area defect specimen, the stress level of the defect can be reduced and the bearing capacity of the specimen can be improved by using the back of the reinforcement and positive reinforcement. Though positive reinforcement effect is superior to the back of the reinforcement, the effects are very limited. Relative to the back of the strengthening, the specimens yield load of positive reinforcement is increased up to 3.5% and the ultimate load is increased up to 2.2%. Therefore, it is proposed that the key application technology of CFRP single side reinforced defect plate: In the practical engineering application, you should first consider the use of positive reinforcement. If positive reinforcement is not possible, you could also consider using the back of the reinforcement.

2. The reinforcement effect is affected by the defect size and depth. The larger the defect area is, the more relaxed the degree of stress concentration of the defect has and the more obvious the effect of front and back reinforcement
is. The deeper the depth of the defect is, the more reduced the level of stress concentration of the defect has and the more obvious strengthening effect is.

ACKNOWLEDGEMENTS
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REFERENCES
STRESS ANALYSIS OF UNBONDED AND BONDED PRESTRESSED CFRP-STRENGTHENED STEEL PLATES

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ABSTRACT

Numerous research studies are available in the literature on strengthening of steel members using carbon fiber reinforced polymer (CFRP) composites. Although it is clear that strengthening using prestressed CFRP plates has several advantages over non-prestressed reinforcements, less attention has been paid in the literature to develop theoretical formulations. Consequently, in this paper, unbonded and bonded prestressed CFRP-strengthened steel plates are analyzed and stresses in CFRP and steel substrate are analytically formulated. Furthermore, the effect of single-side retrofit, which is usually neglected in stress calculations, has been studied. Analytical formulations were then verified by the results of static tests on CFRP-strengthened steel plates under tensile loading. The primary results show that prestressing could considerably reduce the tensile stresses in steel. Consequently, strengthening with prestressed CFRP laminates could be used as an efficient technique for strengthening of steel members, especially those prone to fatigue. However, the available capacity of the prestressed bonded reinforcements before debonding is much less than the developed mechanically anchored, prestressed unbonded reinforcements. Moreover, analytical modeling and experimental results of the current study show that neglecting the eccentricity in single-side CFRP-strengthened steel plates could result in an unsafe stress prediction in steel substrate.

KEYWORDS

Steel member, strengthening, prestressed CFRP, unbonded reinforcement, analytical solution.

INTRODUCTION

Nowadays, a large number of metallic structures are aging worldwide and some of them such as road and railway bridges, offshore structures and communication towers need to be strengthened to carry higher service loads or be safe enough for everyday use due to the fatigue problem. Conventional method of strengthening aging metallic structures, i.e. installing external steel plates to the tension face of the member using bolts or welding, is time-consuming, costly and not so efficient since the external strengthening steel plates are usually bulky, heavy, difficult to fix and prone to fatigue and corrosion of their own (Zhao 2014). On the other hand, unique advantages of carbon fiber reinforced polymer (CFRP) composites such as high corrosion resistance, light weight, high strength, elastic modulus, and fatigue life, made CFRP composites a well-accepted material for static and fatigue strengthening of such structures.

A considerable number of research studies can be found in the literature dealing with strengthening of steel members using CFRP composites (see Zhao and Zhang 2007). Although it is evident that utilizing prestressed CFRP composites has several advantages over non-prestressed reinforcements especially due to a certain decrease in existing stresses in the member, less attention has been paid to model and use prestressed CFRP reinforcements for strengthening of existing steel structures. This might be explained due to the fact that high prestressing levels cannot be reached in bonded CFRP reinforcements. However, this issue can be solved using mechanically anchored, unbonded reinforcements as recently shown by Ghafoori and Motavalli 2015a,b.

The current study presents an analytical solution for stress analysis in unbonded and bonded prestressed CFRP-strengthened steel plates under external tensile loading. To anchor high prestressing forces from CFRP plates to steel substrate a novel prestressed unbonded retrofit (PUR) system has been developed and tested. Afterwards, a set of experiments was performed to evaluate the capability of the analytical model in predicting the stresses in
steel and CFRP as well as the interfacial shear stresses in bonded joints. Note that the limited results, provided in this paper, are part of a large research program on mixed-mode fatigue strengthening of metallic members using CFRP composites at the Swiss Federal Laboratories for Materials Science and Technology (Empa) in collaboration with the Swiss Federal Institute of Technology Lausanne (EPFL) and Monash University in Melbourne.

**ANALYTICAL SOLUTION**

**Unbonded Reinforcements**

In case of utilizing mechanical end anchorages to transfer stresses from CFRP laminates to a steel plate (see schematic configuration in Figure 1a) it is possible to formulate axial stresses in steel and CFRP reinforcements using equilibrium equation as follows:

\[
\sigma_s = T_s - T_{pre} \frac{A_t}{A_s} \left( \frac{1}{E_s} + \frac{1}{E_f} \right) \frac{(\alpha_f - \alpha_s) \Delta T}{A_s}
\]

(1)

\[
\sigma_f = n T_f - \frac{T_{pre} - T_{release}}{A_f} \frac{1}{2A_s} \left( \frac{1}{E_s} + \frac{1}{E_f} \right) \frac{(\alpha_f - \alpha_s) \Delta T}{A_s}
\]

(2)

where \( T_{pre} \) is the initial prestressing force in CFRP laminates, and \( T_{release} \) is the released force which can be calculated as (please refer to the nomenclature for the notations):

\[
T_{release} = \frac{T_{pre}}{1 + \frac{E_s A_f}{E_f A_s}}
\]

(3)

Furthermore, \( n = \frac{E_f}{E_s} \), and \( A_t \) is the transformed cross sectional area of the joint calculated as:

\[
A_t = A_s + 2nA_f
\]

(4)

**Bonded Reinforcements**

**Double-side retrofit**

In case of utilizing CFRP laminates adhesively bonded to a steel plate (Figure 1b), the governing equilibrium equations and boundary conditions can be derived using the free body diagram of an infinitesimal element of the joint depicted in Figure 2 in conjunction with the chosen coordinate systems as shown in Figure 1b. The two differential equations for the horizontal force equilibrium can be expressed as (Täljsten et al. 2009; Albat and Romilly 1999):

\[
\frac{dT_s(y)}{dy} - b_f \tau_s(y) = 0
\]

(5)

\[
\frac{dT_f(y)}{dy} + 2b_f \tau_s(y) = 0
\]

(6)

where \( T_s \) and \( T_f \) are axial forces in CFRP and steel, respectively,

\[
T_s(y) = \sigma_s(y)A_s
\]

\[
T_f(y) = \sigma_f(y)A_f
\]

(7)
The compatibility equations for the CFRP plate, steel, and adhesive can be expressed as:

\[
\frac{d\epsilon_r(y)}{dy} = \frac{T_r(y)}{E_r A_r} - \epsilon_{pre} + \alpha_r \Delta T
\]

\[
\epsilon_r(y) = \frac{dv_r(y)}{dy} = \frac{T_r(y)}{E_r A_r} + \alpha_r \Delta T
\]

\[
\tau_r(y) = G_s \frac{dv_r(y)}{dy} = G_s \tan \left( \frac{v_r(y) - v_s(y)}{t_s} \right) = \frac{G_s}{t_s} (v_r(y) - v_s(y)) + \alpha_r \Delta T
\]

Note that \( b_f \) and \( A_f \) are the summation of widths and cross sectional areas of all individual plates bonded on one side of the steel plate. Differentiating Eq. (10) and substituting Eqs. (8) and (9) gives:

\[
\frac{d\tau_r(y)}{dy} = \frac{G_s}{E_s A_s} \left( \frac{T_r(y)}{E_r A_r} - \epsilon_{pre} + (\alpha_f - \alpha_s) \Delta T \right)
\]

Finally by differentiation of Eq. (11) and substituting of equilibrium equations (i.e., Eqs. (5) and (6)) a second-order ordinary differential equation in terms of interfacial shear stress distribution can be derived as:

\[
\frac{d^2\tau_r(y)}{dy^2} + \lambda \frac{d\tau_r(y)}{dy} = 0
\]

where

\[
\lambda = \frac{G_s}{t_s} \left( \frac{1}{E_s A_s} + \frac{2}{E_f A_f} \right)
\]

Considering the boundary conditions, Eq. (12) leads to the interfacial shear stress as well as axial stresses in the adherents, formulated as follows:

\[
\tau_r(y) = \frac{G_s}{A_r} \left( (\alpha_f - \alpha_s) \Delta T - \epsilon_{pre} - \frac{T}{E_r A_r} \right) \sinh (\lambda y)
\]

\[
\sigma_r(y) = \frac{G_s}{A_f} \lambda \frac{T}{E_f A_f} \left( (\alpha_f - \alpha_s) \Delta T - \epsilon_{pre} - \frac{T}{E_r A_r} \right) \left( \frac{\cosh (\lambda y)}{\cosh (\lambda l)} - 1 \right)
\]

\[
\sigma_s(y) = \frac{T}{A_s} \frac{2G_s}{\lambda^2 A_f} \left( (\alpha_f - \alpha_s) \Delta T - \epsilon_{pre} - \frac{T}{E_r A_r} \right) \left( \frac{\cosh (\lambda y)}{\cosh (\lambda l)} - 1 \right)
\]

where \(-l \leq y \leq l\).

**Single-side retrofit**

In the most practical cases, both sides of the considered steel plate (which is assumed to be a part of a steel structure) are not accessible. As a consequence, it is instructive to derive the governing equations for single-side CFRP-strengthened steel plates. In this case the steel plate undergoes a bending moment due to the eccentricity of the system. Furthermore, it is obvious that in this case the solution is dependent on the general boundary condition of the steel plate. Through the following solution, it is assumed that the whole system is simply supported. The two differential equations for the horizontal force equilibrium can be expressed as:

\[
\frac{dT_r(y)}{dy} - b_f \tau_r(y) = 0
\]

\[
\frac{dT_r(y)}{dy} + b_f \tau_r(y) = 0
\]

where \( T_r \) and \( T_s \) are axial forces in CFRP and steel, respectively (see Eq. (7)). The compatibility equations for the CFRP, steel, and adhesive can be expressed as:

\[
\epsilon_r(y) = \frac{dv_r(y)}{dy} = \frac{T_r(y)}{E_r A_r} - \epsilon_{pre} + \alpha_r \Delta T
\]

\[
\epsilon_s(y) = \frac{dv_s(y)}{dy} = \frac{T_s(y)}{E_s A_s} + \frac{M_s(y) t_s}{2E_s I_s} + \alpha_s \Delta T
\]

\[
\tau_s(y) = G_s \frac{dv_s(y)}{dy} = G_s \tan \left( \frac{v_s(y) - v_f(y)}{t_s} \right) = \frac{G_s}{t_s} (v_s(y) - v_f(y))
\]

Differentiating Eq. (21) and substituting Eqs. (19) and (20) gives:
Differentiation of Eq. (22) yields:

$$\frac{d^2\tau_s(y)}{dy^2} = \frac{G_b}{t_s} \left[ \frac{dT_f(y)}{dy} - \frac{dT_f(y)}{dy} - \frac{dM_s(y)}{dy} - \frac{M_s(y)}{2E_sI_s} \right]$$

The moment equilibrium of the differential segment of the steel plate gives:

$$\frac{dM_s(y)}{dy} + \frac{b_t}{2} \tau_s(y) = 0$$

Finally the differential equation governing the interfacial shear stress distribution can be derived by substituting of force equilibrium equations (Eqs. (17) and (18)) as well as the moment equilibrium equation (Eq. (24)) in Eq. (23) as:

$$\frac{d^2\tau_s(y)}{dy^2} - \lambda^2 \tau_s(y) = 0$$

where

$$\lambda^2 = \frac{G_b \left( \frac{1}{E_sA_t} + \frac{1}{E_sA_t} + \frac{t_s^2}{4E_sI_s} \right)}{t_s}$$

Solving the differential equation given in Eq. (25) and applying the boundary conditions leads to the interfacial shear stress as well as axial stresses in the adherents, formulated as follows:

$$\tau_s(y) = \frac{G_b}{2t_s} \left( (\sigma_s - \sigma_a)\Delta T - \sigma_{pre} - \frac{T}{E_sA_t} \right) \left( \sinh (\lambda y) \right) \cosh (\lambda l)$$

$$\sigma_s(y) = \frac{G_b}{\lambda^2 I_s} \left( (\sigma_s - \sigma_a)\Delta T - \sigma_{pre} - \frac{T}{E_sA_t} \right) \left( \cosh (\lambda y) \right) \cosh (\lambda l) - 1$$

$$\sigma_a(y) = \frac{T}{A_t} b \frac{G_b}{\lambda^2 I_s} \left( (\sigma_s - \sigma_a)\Delta T - \sigma_{pre} - \frac{T}{E_sA_t} \right) \left( \cosh (\lambda y) \right) \cosh (\lambda l) - 1$$

where \(-l \leq y \leq l\).

### EXPERIMENTAL PROGRAM

### Test Layout and Material Properties

In order to evaluate the performance of the developed analytical model in predicting CFRP and steel contributions in a CFRP-strengthened steel plate under uniaxial tensile loading, a set of experiments on steel plate specimens with dimensions of 1100×150×10 mm (length×width×thickness) was planned. The test layout and specimens’ description are provided in Table 1. The utilized steel was of type S355J2+C with nominal yield strength of 355 MPa. The Young’s modulus of the steel was determined equal to 200.9 GPa. Furthermore, normal modulus unidirectional CFRP plates of type S&P 150/200 with cross sectional dimensions of 50×1.4 mm (width×thickness) was used. The elastic modulus and ultimate strength of the laminates were measured as 156 GPa and 2905 MPa, respectively.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Specimen description</th>
<th>Adhesive thickness (mm)</th>
<th>Prestressing force (kN)</th>
<th>Strain in steel at 300 kN (µs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>reference</td>
<td>-</td>
<td>-</td>
<td>995</td>
</tr>
<tr>
<td>2</td>
<td>bonded_non-prestressed_single-side</td>
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</tr>
<tr>
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<td>970</td>
</tr>
<tr>
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<td>-</td>
<td>925</td>
</tr>
<tr>
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<td>120</td>
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</tr>
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<td>unbonded_prestressed_double-side</td>
<td>-</td>
<td>120</td>
<td>547</td>
</tr>
</tbody>
</table>

Table 1 Specimens’ description and test results
In those specimens strengthened using adhesively-bonded CFRPs, a two component epoxy adhesive of type Araldite 420 was used. The thickness of adhesive was measured after curing process using a micrometer and the average values for each of the specimens are provided in Table 1. The ultimate tensile strength of the adhesive was determined equal to 26.0 MPa obtained from three 6-day samples (see Michels et al. 2016 for further info on the test process). No auxiliary test was performed to determine shear modulus of the adhesive, $G_s$, as part of the current study; however, $G_s=730$ MPa was used in the analytical model based on the manufacturer’s catalogue.

**Strengthening Procedure**

In order to strengthen steel plate specimens using bonded CFRP plates, first the bond area was sand-blasted and carefully cleaned with acetone. Two 520-mm-long CFRP plates were glued to one side of the specimen No. 2, while in specimen No. 3, one CFRP strip was used on each side. The specimens were cured in the room temperature for 6 days before testing. On the other hand, strengthening of the steel plates using mechanical unbonded clamps was much faster since the clamps work based on friction and no surface preparation for steel plate neither curing for adhesive is required. Each set of the designed clamps consists of a hard plate which is in contact with the CFRP plate, pressing it to the steel substrate using the prestressing force generated in M12 bolts (grade 12.9). In total 8 bolts were used in each of the clamps, tightened with a torque of 160 N.m, generating 72 kN prestressing force per bolt. As the whole system works based on friction, diamond friction foils (see 3M technical catalogue) was used between the CFRP and steel. Specimen No. 4 was easily prepared by putting different parts of the mechanical clamps together and tightening the M12 bolts of each clamps, which tied the CFRP plates to the steel substrate. It should be mentioned here that due to the lack of space, no detailed info regarding the design of mechanical clamps could be provided as part of this paper.

To strengthen steel plate specimens No. 5 and 6, an especial prestressing setup was designed to simultaneously prestress two parallel CFRP laminates (Figure 3). The distance between the two CFRPs was accurately kept equal to 10.40 mm using high strength steel cubes in the prestressing grips. The laminates were prestressed up to a load level of 120 kN (approx. 30 % tensile capacity of the laminates) using a hydraulic jack, while the load and strain in CFRPs were monitored using a 300 kN load cell and strain gauges applied on the CFRPs, respectively. After prestressing the laminates, epoxy adhesive was applied on the prepared surfaces of specimen No. 5 and the specimen was put in between the strips. The prestressing force was kept constant for 6 days, and afterwards released to zero. On the other hand, specimen No. 6 was prepared almost in 1 hour due to the fact that after prestressing, CFRP laminates were immediately clamped to the specimen using mechanical anchorages (Figure 3). After releasing the prestressing force, CFRP laminates were cut from both sides of the prestressing grips and the strengthened specimens were carried to the testing machine.

**Test Setup**

All the specimens were equipped with conventional foil strain gauges on steel and CFRPs, as well as magnetic strain gauges on steel. The specimens were tested under displacement-controlled condition with a speed of 1 mm/min using a 1000 kN static-fatigue servo-hydraulic Schenck machine with an Instron controller. Test setup and the typical bonded and unbonded CFRP-strengthened specimens are shown in Figure 4.
RESULTS AND DISCUSSIONS

As it was mentioned before, fatigue strengthening of metallic members is the main focus of the current ongoing research project. Consequently, all the reference/strengthened specimens were subjected to uniaxial tension up to a load level equal to 300 kN, which corresponds to approximately 50% of yield strength of the bare steel. Except test No. 5 in which both of the CFRP plates were debonded from the steel plate at load levels of 50 and 153 kN, respectively, neither any interface failure in bonded reinforcements nor any slip between CFRP and steel in unbonded reinforcements occurred in all other specimens. Load vs. strain in the middle part of the steel specimens were plotted in Figure 5, while the ultimate strain values in steel are also provided in Table 1 for comparison. As expected, strengthening of steel plates using bonded/unbonded CFRP reinforcements increases the stiffness of the member while stiffness of the single-side retrofit is relatively lower than that of double-side strengthening due to the eccentricity of the system. Furthermore, Figure 5 clearly shows the advantage of prestressing in reducing the strains in the steel substrate. However, as it was previously mentioned, in test specimen No. 5 the prestressed CFRP plates were debonded from the steel plate in low load levels, while the mechanical clamps could carry the prestressing force up to the end of loading procedure. It is worth mentioning here that specimen No. 6 was loaded up to 800 kN to further investigate the performance of the designed PUR system. It was observed that no slip between the prestressed CFRP plates and steel substrate occurred even at a load level of 800 kN, which proves the high performance of the designed PUR system.
Load-strain behavior of strengthened specimens was predicted by the proposed analytical formulations and the predictions are also plotted in Figure 5 for comparison purposes. It can be concluded from Figure 5 that the analytical model is capable of accurately predicting the behavior of non-prestressed and prestressed CFRP-strengthened steel plates. It is obvious that linear elastic behavior of all the individual elements of the joint (i.e. steel, CFRP, and adhesive) is a pre-assumption of the developed analytical model, which is also the case in fatigue problem. It means that the model is only applicable up to the end of the elastic regime in steel.

In specimen No. 3, seven strain gauges were mounted on half-length of the CFRP plate at distances of 10, 30, 50, 70, 90, 175, and 260 mm from the CFRP end (see Figure 4a). Utilizing the set of strain gauges, the evolution of interfacial shear stresses in the CFRP-to-steel bonded joint was determined and the results were used to construct the data points presented in Figure 6 (please refer to Hosseini and Mostofinejad 2013 for the calculation of interfacial shear stresses based on the CFRP strain measurements). Moreover, distribution of the interfacial shear stresses along the bond length, predicted by Eq. (14) was also plotted for different load levels in Figure 6. Comparing the experimental data points with the proposed analytical solution shows the high capability of the model in predicting interfacial shear stress distribution along the bond length. Numerous research studies on bond behavior of CFRP composites to concrete/steel substrate proved the fact that not the whole bond length contributes in load bearing, and there exists an effective bond length, which the interfacial shear stresses are transferred to the substrate through this zone (Yu et al. 2012; Fernando 2010; Hosseini and Mostofinejad 2014). Experimental results, presented in Figure 6, reveal that the effective bond length of the utilized bonded joint was about 40 mm. The distribution of interfacial shear stresses, obtained from the analytical model, shows that in the middle 440 mm of the bond length (220 mm from the centerline on each side) there is almost no interfacial shear stress. Consequently, the model can be also used to have an approximation regarding the effective bond length of adhesively bonded CFRP reinforcements.

CONCLUSIONS

Based on the analytical modeling and experimental tests done in the current study, the following concluding remarks can be drawn:

- A very good correlation was observed between the results of the developed analytical model and experiments performed on CFRP-strengthened steel plates under uniaxial tension. The model can be used to accurately estimate stresses in CFRP and steel substrate both in case of non-prestressed and prestressed CFRP laminates bonded or mechanically anchored to the steel substrate in the elastic regime. Interfacial shear stresses, and subsequently, the effective bond length of CFRP-to-steel can be estimated using the developed analytical model.
- Experimental results of the current study showed that even though adhesively bonded CFRPs can transfer prestressing loads to the steel substrate, the available capacity of the prestressed bonded laminates would be very low for carrying external loads, and the CFRP plate would be debonded from the steel substrate in low load levels.
- A mechanical anchorage system is developed to transfer high prestressing forces to the steel substrate via friction. Experimental tests on steel plates strengthened with unbonded CFRP laminates using the aforementioned system revealed the high capacity of the system, since no slip/failure was occurred in those specimens. Furthermore, despite in bonded joints, no special surface preparation neither curing is required.
when mechanical clamps are used. Consequently, strengthening steel members using prestressed unbonded CFRPs can be a very efficient technique, especially in cases that a reduction in existing stresses is needed such as in fatigue strengthening of steel bridges.

- Both results of the analytical modeling and experimental tests revealed that neglecting the eccentricity in single-side CFRP-reinforced steel members, might lead to an unsafe prediction of stress level in steel, especially when the steel member is relatively thick.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the financial support provided by the Swiss National Science Foundation (Project No: 5211.00892.100.01). The authors would like to thank the technicians of the Structural Engineering Research Laboratory and Mechanical Systems Engineering Laboratory of Empa for their cooperation in performing the experiments. Excellent support from S&P Clever Reinforcement Company AG, Switzerland, for providing the materials of the current study is highly appreciated.

NOMENCLATURE

<table>
<thead>
<tr>
<th>Latin symbols</th>
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<tr>
<td>A</td>
<td>Area</td>
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<tr>
<td>b</td>
<td>Width</td>
</tr>
<tr>
<td>E</td>
<td>Young’s modulus</td>
</tr>
<tr>
<td>F</td>
<td>Force</td>
</tr>
<tr>
<td>G</td>
<td>Shear modulus</td>
</tr>
<tr>
<td>I</td>
<td>Moment of inertia</td>
</tr>
<tr>
<td>l</td>
<td>Length</td>
</tr>
<tr>
<td>M</td>
<td>Bending moment</td>
</tr>
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<td>n</td>
<td>Elastic modulus ratio</td>
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<td>t</td>
<td>Thickness</td>
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<tr>
<td>T</td>
<td>Tension force or resultants</td>
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<td>pre</td>
<td>Prestressing</td>
</tr>
<tr>
<td>s</td>
<td>Steel</td>
</tr>
<tr>
<td>t</td>
<td>Transformed section</td>
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</table>

Thermal expansion coefficient
Strain
Elastic shear stress distribution parameter
Longitudinal displacement
Normal stress
Shear stress

Subscripts

REFERENCES


Manufacturer catalogue, 3M Technical Ceramics. 3Mfriction shims.. 3M Deutschland GmbH.


ABSTRACT

This study addresses behavior of bond between fiber reinforced polymer (FRP) and steel plate. A total of 48 FRP-steel double-lap bond specimens, subjected to elevated temperature ranging from room temperature to 120 °C, were tested to investigate the performance of the bond between FRP and steel. The experimental results showed that the failure mode of test specimens was altered from steel-adhesive debonding to FRP-adhesive debonding. The load carrying capacity decreased significantly at temperatures approaching or exceeding the glass transition temperature ($T_g$) of adhesive. Based on the experimental results, a model is proposed to analyze the FRP-steel interface at elevated temperature.

KEYWORDS

Bond, FRP, steel plate, elevated temperature, ultimate load.

INTRODUCTION

Fiber-reinforced polymer (FRP) has been widespread implemented in upgrading structures due to its preferable mechanical properties, including high strength, light weight, good fatigue resistance and flexibility in shape. In the repair of steel structures with FRP method, the FRP is bonded to the steel surface using epoxy adhesives. Both the flexural and the shear strength of steel structure can be significantly increased, and adequate overall ductility can be obtained. As the load is transferred to the FRP material by epoxy adhesives, the durability of its bond with steel is critically important for the safety of the repaired structure.

Although a few researches have been conducted to better understand the bonding performance of FRP-steel interface at ambient temperature (Fawzia et al. 2006, 2010), very few studies have been performed to evaluate the bond behavior between FRP and steel plate subjected to temperature, which is particularly important for building applications where hot climate or fire is of high concern. Most epoxy polymers are vulnerable to high temperatures and their mechanical properties will decrease rapidly when the temperature exceeds $T_g$, at which they transform from glassy state to rubbery state. The behavior could lead to the reduction in mechanical properties of both adhesive and FRP composite, weaken the stress transfer, resulting in failure in adhesively-bonded joints. This study investigates the characteristics of FRP-steel bond under temperature.

EXPERIMENTAL PROGRAM

Test Specimens

A total of 48 double-lap shear bond specimens, divided into 16 groups of three identical samples, were tested. The experimental parameters selected were FRP type (CFRP and GFRP) and the temperature ($T$ = 27, 40, 50, 60, 70, 80, 100, 120 °C) respectively. The detail of specimens is shown in Figure 1. Two FRP sheets were bonded
symmetrically to the 3mm-thickness steel plate surface over a length of 150 mm. A two-part epoxy adhesive with the \( T_g \) being 50 °C was applied for all test specimens. The mechanical properties of adherends and adhesive are listed in Table 1.

<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness (mm)</th>
<th>Young's Modulus (GPa)</th>
<th>Yield Strength (MPa)</th>
<th>Ultimate Strength (MPa)</th>
<th>Failure Strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel plate</td>
<td>3</td>
<td>205.0</td>
<td>350.0</td>
<td>451.0</td>
<td>20.0</td>
</tr>
<tr>
<td>CFRP</td>
<td>0.111</td>
<td>252.0</td>
<td>-</td>
<td>3553.0</td>
<td>1.4</td>
</tr>
<tr>
<td>GFRP</td>
<td>0.169</td>
<td>109.0</td>
<td>-</td>
<td>2301.0</td>
<td>2.7</td>
</tr>
<tr>
<td>Epoxy resin</td>
<td>-</td>
<td>( \geq 2.5 )</td>
<td>( \geq 30.0 )</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

a Manufacturer data

**Test Setup and Instrumentation**

For control specimens, nine strain gauges were attached on the top of the FRP sheet spacing at 20 mm. For specimens at elevated temperatures, strain gauging with adhesively bonded strain gauges is difficult due to the susceptibility of the adhesive to heat. The displacements calculated from strain readings basically agree well with the LVDT readings reported by Yu et al. (2012). Therefore, two L-shaped cooper plates, which had a low coefficient of thermal expansion \( (1.65 \times 10^{-5}/°C) \) were attached symmetrically for measuring displacement at the loaded end. Specimens were preheated for 20 minutes in a device and kept inside throughout the testing process. The tests were conducted using a universal testing machine with a capacity of 100 kN. The schematic view of test setup is shown in Figure 2.

![Figure 2 Sketch of test setup](image)

**EXPERIMENTAL RESULTS AND DISCUSSION**

**Failure mode**

A shift on the failure mode from steel-adhesive debonding to FRP-adhesive debonding was observed. Both CFRP series and GFRP series showed similar failure modes, and thus only one set of images was provided in Figure 3. In the range of 27-50 °C, specimens displayed a steel-adhesive debonding failure. The debonding initiated from the loaded end and then heading to the free end along the steel surface. The results suggest that, at temperatures below the \( T_g \), the adhesive used is brittle, glassy and has microcracking propensity, flaws and imperfections. This in turn will weaken the interfacial adhesive layer, lead to the debonding failure between steel and adhesive (Al-Shawaf et al. 2009). In the range of 60-120 °C, the debonding failure occurred along the interface between FRP and adhesive, with a few of adhesive left on the steel plate. In addition, with the increase of temperature, more residual epoxy was attached on the surface of steel plate. This failure shift could be attributed to the degradation of bond-strength in adhesion at elevated temperature.

![Figure 3 Failure modes of specimens](image)

**Analytical Modeling**

For specimens at ambient temperature, based on the bilinear model proposed by Dai et al. (2005), a model adaptive to predict the relationship of shear stress and interfacial slip is developed.
As shown in Figure 4, the equation of equilibrium for the FRP sheet can be written as:

\[
\frac{d\sigma_f(x)}{dx} - \frac{\tau(x)}{t_f} = 0
\]  

(1)

The relationship of the FRP strain and interfacial slip should be known to obtain the \( \tau-s \) curve. According to the current literature review (Yuan et al. 2004; Xia and Teng 2005; Dai et al. 2006; Yu et al. 2012; Dai et al. 2014), the bond-slip curves for FRP-to-steel bonded joint are similar to those for FRP-to-concrete bonded joint, both have a bi-linear shape. And their development of shear stress are the same. Moreover, the strain-slip curves at the loaded end derived from the load-slip relationship for FRP-to-steel bonded joints are similar to those for FRP-to-concrete bonded joints. Therefore, a unique expression linked to an exponential term is proposed according to reference (Dai et al. 2006, 2014):

\[
\varepsilon(x) = f(s(x)) = \alpha (1 - e^{-\beta s})
\]  

(2)

where \( \alpha \) and \( \beta \) are regression parameters.

Combining Eqs 1 and 2, and neglecting the strain of steel plate, the bond stress versus interfacial slip can be expressed as:

\[
\tau = t_f \frac{d\sigma_f(x)}{dx} = E_f t_f \frac{d\varepsilon(x)}{dx} = E_f t_f \frac{df(s(x))}{ds(x)} \frac{ds(x)}{dx} = \alpha^2 \beta E_f t_f e^{-\beta s} (1 - e^{-\beta s})
\]  

(3)

where \( E_f \) and \( t_f \) are the tensile modulus and the thickness of FRP sheet respectively.

Therefore, the maximum load, \( P_{\text{max}} \), transmitted to the FRP sheet can be expressed as:

\[
P_{\text{max}} = \varepsilon_{\text{max}} b_f t_f E_f = \lim_{s \to \infty} \alpha (1 - e^{-\beta s}) b_f t_f E_f = \alpha b_f t_f E_f
\]  

(4)

For specimen at elevated temperature, the effect of elevated temperature is to degrade the mechanical properties of epoxy-impregnated FRP composite, especially when the temperature exceeds the glass transmission temperature (\( T_g \)). According to the degradation model by Mouritz and Gibson (2006), the relationship of FRP tensile modulus and temperature can be written as:

\[
E_f(T) = \frac{E_U + E_R}{2} - \frac{E_U - E_R}{2} \tanh[k(T - T_g)]
\]  

(5)

in which, \( E_U \) and \( E_R \) are the unrelaxed (low temperature) and relaxed modulus (high temperature) respectively; \( k \) is a constant; \( T_g \) is the temperature when a 50% reduction in property value is observed.

And the ultimate load at the loaded end with enough bonded length can be written as:

\[
P_{\text{max}} = E_f(T) t_f b_f \left[ \alpha - (\alpha_f - \alpha_s)\Delta T \right]
\]  

(6)

where \( \alpha_f \) and \( \alpha_s \) are the linear coefficients of thermal expansion for FRP and steel respectively, \( \Delta T \) is the service temperature variation (thermal loading).
Figure 5 reports the comparison of experimental results and predictions using the proposed model. It should be noted that a few ultimate loads were not recorded due to the premature failure of specimens at high temperature. As can be seen, good agreement between predicted and experimental values are observed in all specimens regardless of FRP type. In the range of 27-50 °C, the average strengths of specimens showed a progressive decrease as the temperature increased, and the average ratios of predictions to experimental data are 1.019 and 1.012, with the coefficients of variation (COV) being 0.022 and 0.069 respectively. When the temperature exceeds 50 °C, a significant reduction of ultimate load was observed, and a few specimens exhibit lower bond strength than the prediction. The reason for this phenomenon was that probably the internal stress created within the interface due to the volatilization of decomposition as a result of adhesive volume decrease, together with the thermal stress caused by differences of the coefficients of linear thermal expansion of the adhesive and the adherends, would increase the porosity and flaws within the adhesive which, in turn, have a detrimental effect on adhesive-joint capacity.

CONCLUSIONS

The failure mode of specimen was affected by the experimental temperature. It was likely to change from steel-adhesive debonding to FRP-adhesive debonding. The load carrying capacity of specimens degraded with the increase of temperature. This degradation was pronounced at temperatures above $T_{c}$. Based on the results, a model is proposed for analyzing the bond behavior between FRP and steel at elevated temperature.

ACKNOWLEDGMENTS

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TEST ON FLEXURAL PERFORMANCE OF RC BEAMS STRENGTHENED WITH PRESTRESSED HB-CFRP SYSTEM

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Geotechnical and Structural Engineering Research Center of Shandong University, Jinan 250061

ABSTRACT

By comparing and analyzing the flexural performance of reinforced concrete (RC) beams strengthened with HB-CFRP system and prestressed HB-CFRP system, the influence of prestress on the flexural performance of the RC beams was investigated and the flexural capacity, CFRP strain, mid-span deflection and the utilization of the CFRP material strength were investigated. The test results showed that this strengthening technique which RC beams are strengthened with using prestressed HB-CFRP system can restrain effectively the formation and development of cracks, improve the stiffness of the beam. Compared with non prestressed method of HB-CFRP system, the cracking load of the beam is increased by 71% and the flexural capacity is increased by 67%. The strengthening technique can also restrain the concrete stripping, avoid the debonding failure of CFRP and improve the utilization of the CFRP material strength.

KEYWORDS

HB-CFRP system, prestressed CFRP plate system, reinforced concrete beam, text on flexural performance, strengthening, flexural capacity.

INTRODUCTION

Fiber Reinforced Polymer (FRP) is widely used to the reinforcement of Reinforced Concrete (RC) structures and other structures (Ying-ming LI et al. 2009; Wang Sheng-yi 2005). Especially, external bonding of FRP plates (EB-FRP) has became as a simple, popular and flexible method for the reinforcement of RC beams (Lawrence Cbank et al. 2007; Li Song-hui et al. 2005; Nanni A 1997). However, the strengthening method of EB-FRP system has obvious defect that is the premature debonding of the composite laminates (Teng et al. 2005; Yang Yong-xin et al. 2004). So the utilization of FRP material strength is only 15% ~ 35% (Yan Jun-hui 2009). Besides, the failure mode of the EB-FRP system tends to be brittle failure (Guo Zhang-gen et al. 2007). As a result, City University of Hong Kong develops a kind of strengthening technique of RC beams with hybrid bonded FRP. The combination of EB-FRP and mechanical anchorage improves the bonding performance between FRP and concrete. The experimental study in the paper (Wu Yu-fei et al. 2008) shows that the interfacial bond strength of HB-FRP system is 7.5 times higher than that of EB-FRP system.

However, the HB-FRP system without using prestress still has obvious shortcomings. It is observed that the FRP does not work until the reinforced structure is loaded again. The efficiency of the strengthening method using HB-FRP system is too low. This method is not able to adjust the internal force of structure; it also has little effect on modifying the mechanical performance of reinforced structure (Ti An-guo 2006). Compared with the non prestressed HB-FRP system, the prestressed HB-FRP system can improve the utilization of the CFRP material strength, save the 30% ~ 50% FRP material usage and avoid the deboning of the FRP plates (Wang Xing-guo et al. 2005; Lin Yu-dong et al. 2013). Further more, the prestressed HB-FRP system can restrain the formation and development of cracks. Therefore, more and more people are interested in studying the prestressed HB-FRP system, so the corresponding experimental and theoretical study of RC beams are carried out. Abroad, Deuring (Deuring M 1993) carried out the experimental study on flexural behaviour of large proportion of the T Beams strengthened with non prestressed or prestressed FRP plates. Garden, Quantrill R J, Dong-Suk Yang, Abhijit Mukherjee et al. (Garden H N et al. 1998; Quantrill R J et al. 1998; Dong-Suk Yang et al. 2009; Abhijit Mukherjee et al. 2009; Hajishahemi A et al. 2011) carried out the experimental study on flexural behavior of the corresponding model and the prototype beams. In China, Tian An-guo, Wang Xing-guo, Lin Yu-dong et al. (Tian An-guo 2006; Wang Xing-guo et al. 2005; Lin Yu-dong et al. 2013) also carried out the experimental and theoretical study of RC beams strengthened with the prestressed FRP and obtained some research results.
In addition, He Xue-jun, Zhou Chao-yang et al. (He Xue-jun et al. 2008) carried out experimental study on the flexural behavior of RC beams strengthened with near-surface mounted CFRP laminates. The near-surface mounted CFRP laminates can prevent the FRP from peeling off more effectively and has more flexural capacity than the externally bonded CFRP-strengthening beams. Based on the concept of failure mechanics and considering interfacial shear stress transfer, effective stress transfer length and no interfacial debonding failure, Niu He-Dong et al. (Niu He-dong et al. 2002) carried out the theoretical derivation of the maximum possible prestress on bonding reinforcement technique of prestressed FRP laminates.

The focus of this paper is to study the flexural performance of RC beams strengthened with prestressed CFRP plates and mechanical anchorage. Two beams including HB-CFRP system reinforced beam and prestressed HB-CFRP (PHB-CFRP) system reinforced beam were subjected to flexural test. Compared with HB-CFRP reinforced beam, the flexural capacity, failure mode and load-deflection behavior of PHB-CFRP reinforced beam were taken as the main research parameters.

EXPERIMENTAL PROGRAM FOR RC BEAMS STRENGTHENED WITH PHB-CFRP SYSTEM

Beam specimen Design

In order to explore the flexural behavior of RC beams strengthened with PHB-CFRP system, two beams are subjected to flexural test. Flexural test was conducted on a control beam strengthened with HB-CFRP system, on a beam strengthened with PHB-CFRP system. The main experimental parameter is whether or not the prestress is applied. RC beams were cast with dimensions of 3300mm (l)×240 mm (b)×350 mm (h). Specifications of steel include longitudinal reinforcement (4φ20) and stirrup (φ8@150). The characteristics of all the beams, as well as their steel reinforcement details, are shown in Figure 1 and Table 1. Concrete compressive strength is 30MPa. The width and thickness of CFRP plate is 50mm and 3mm respectively. The ultimate strength of CFRP plate is 2.8GPa. The construction method of prestressing CFRP beam uses similar post-tensioning method. In the test, the special anchorage and clamp (Figure 2) were applied to anchoring the end of CFRP plate. In addition, the steel plate and chemical bolts were used to anchor the CFRP at the bottom of the beams. The size of the steel plate is 120mm×60mm. The chemical bolt is a kind of expansion bolt. The diameter of the chemical bolt is 10mm, the length is 57mm, and the anchorage depth is 50mm (Figure 3).

![Figure 1 Beam details (units: mm)](image1)

(a) Prestressed anchor device

(b) Anchor clamp

Figure 2 Anchoring device and clamp for PHB-CFRP reinforced beam

![Figure 3 Steel fastener details (units: mm)](image2)

Table 1 Parameters of bending test specimens

<table>
<thead>
<tr>
<th>Number</th>
<th>Reinforcement method</th>
<th>Initial stress /MPa</th>
<th>Anchor spacing of steel fasteners/mm</th>
</tr>
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<tr>
<td>HB</td>
<td>HB-CFRP reinforcement</td>
<td>-</td>
<td>300</td>
</tr>
<tr>
<td>PHB</td>
<td>Prestressed HB-CFRP reinforcement</td>
<td>333.3</td>
<td>300</td>
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</tbody>
</table>
Beam specimens Making
HB-CFRP beam specimen detail

HB-CFRP test beam uses wet bonding method for the FRP plate and the steel plates - expansion bolts for reinforcement. First of all, the surface of the beam was cleaned which the loose concrete was removed by using the grinding machine, so the solid concrete aggregate was exposed. The area of concrete which is used to bond CFRP plate should be chiseled. Then the epoxy adhesives was put on that the place of the beam. Second the dipping adhesive was applied to bond the surface of the concrete and the corresponding surface of the CFRP plate. The dipping adhesive was uniformly distributed. Then put the CFRP plate on the surface of the concrete and used the special rubber roller to uniformly roll the composite material, squeezed the bubble, so that the surface between concrete and CFRP could be completely bonded. Finally the positions of the reserved holes of the steel fasteners were needed to setting out. Meantime we used drilling rig to drill holes and used a high pressure air gun to clean up. Then the expansion bolts were put into the reserved holes, the mechanical anchoring steel plates were installed, and the nuts should be tightened by use of the torque wrench (Figure 4). The contact surfaces between the mechanical anchoring steel plates and the CFRP plates were needed to apply epoxy adhesives to reinforce. After the reinforced system was completed, the RC beam could be tested. The beam specimen strengthened with HB-CFRP plates is shown in Figure 5.

PHB-CFRP beam specimen detail

The manufacturing process of the PHB-CFRP beam includes the manufacturing process of HB-CFRP beam as well as the application of prestress. Before pouring concrete, the bolts of the pedestal should be embedded. Firstly, the CFRP plate was bonded on the surface of the concrete liking traditional EB-FRP system. Then the fixed end anchor and the tensioning end anchor were located in the beam and the counterforce screws were gone through the tensioning end anchor, stretching tighten the lock nuts. Secondly the jack was applied to give the force, and the pressure sensor was used to measure the reaction force. When the loading force reached the required value, the anchor nuts were tightened. Finally, the CFRP plate was strengthened through the anchorage system consisting of the steel plate and the chemical bolts. After the structural adhesive was fully solidified, the loading device could be removed to carry out the flexural test. The loading device of prestress is shown in Figure 6 and the anchorage system of the CFRP plate is shown in Figure 7.

Experimental Design

The test beams are simply supported at both ends. The tests are based on mid-span static loading and are carried out on the 100t counterforce frame. The loading system is shown in Figure 8.
Experimental parameters include the load size, deflection, strain of CFRP. The size of the load was controlled by the pressure sensor during the test. Arrangement of the strain measuring points and the deflection measuring points of the two beams was same. The strain measuring points were arranged in the longitudinal 2m range of the FRP plates and at the bottom of the concrete beams. The deflection measuring points were arranged in the bottom of support, 1/4 span and 1/2 span. During the loading process, high precision electronic dial gauges were used to measure the deformation of the beams. The strain and deflection measuring points are shown in Figure 9.

EXPERIMENTAL PHENOMENON

The concrete in the place of loading point is crushed. In the tests, it didn’t occur CFRP tensile failure. During the process of loading, the interface between CFRP and concrete came out "crackling" sounds. It was observed that the bonding interface began to occur debonding. When the load could no longer continue to increase, the debonding failure length and distribution of the CFRP plate of the PHB-CFRP beam was found to be smaller than that of the HB-CFRP beam. In addition, the cracking load size of the PHB-CFRP beam was much greater than that of the HB-CFRP beam. Due to the existence of prestress, the formation and development of the cracks of the RC beam are effectively restrained and crack spacing and crack length distribution become smaller. The cracks of the HB-CFRP beam and the PHB-CFRP beam are extended to the top of the beams. The longitudinal cracks distribution of the HB-CFRP beam are uniform and the mostly cracks are vertical. Compared with this phenomenon, the PHB-CFRP beam has a large number of oblique cracks. In the end, there are two of the "eight" - shaped oblique cracks (Figure 10) which are from the bottom of the beam to the loading point, so that the load size can not continue to rise.
RESULTS AND DISCUSSION
Fracture characteristics and Load-carrying capacities

In the experiment, the formation and development of the cracks were continuously observed. When the HB-CFRP beam was loaded to the 35.2kN, the first vertical crack occurred in the mid-span section, and the corresponding cracking load of the PHB-CFRP beam was raised to 60.1kN. Compared with the former, the cracking load was increased by about 71%. Test results show that the PHB-CFRP reinforcement can effectively restrain the appearance of cracks and increase the stiffness of the RC beam. When the load size is loaded to 70kN, the distribution of cracks (Figure 11) of the two beams are observed and recorded. It is found that the extended height and longitudinal range of the cracks of the PHB-CFRP beam are smaller. As a result, it shows that the beam strengthened with PHB-CFRP system can effectively restrain the expansion and extension of cracks. When loading to the failure of the beams (Figure 12), the flexural capacity of HB-CFRP beam is about 120kN and that of PHB-CFRP beam is about 200kN. Obviously, the flexural capacity of the PHB-CFRP beam is increased by about 67%.

FRP strain distribution

In the experiment, the strain distribution of CFRP at the bottom of the testing beams is shown in Figure 13 and Figure 14. It can be seen that the CFRP strain distribution is approximately symmetrical, which the maximum value is in mid-span section, and minimum value is at the end of beam. When the load size is small, the FRP strain increases uniformly with the increase of the load size. When the load is up to 130kN, the strain in the 1.4 m length range from the left end to the right section of HB-CFRP beam bottom decreases suddenly. This is because the left end of CFRP plate occurs debonding. The 0.4m length range of the right section of the CFRP plate does not occur debonding, so that the strain changes a little compared with the former stage. The ultimate strain value of CFRP plate is 17500με. The maximum strain value of HB-CFRP beam is 2580με, the material strength utilization is 14.7%. The PHB-CFRP beam’s initial strain value is 2080με. While the PHB-CFRP beam is loaded to failure, the maximum strain value of CFRP plate is 5406με, so the material strength utilization is 30.9%. The experiment shows that the utilization of CFRP material strength can be improved obviously by using prestress.

Load-deflection curves

The load-deflection curves in mid-span section of the test beams are shown in Figure 15, and that of the left and right 1/4 section are shown in Figure 16. The load-deflection curves are the shape of two sections, the maximum deflection value is in mid-span, and the two sides of the deflection distribution of the beam is approximately symmetrical. When the load size is increased to A point, the load increment size begins to decrease and the deflection size increase rapidly. In the initial stage, the load-deflection curves of HB-CFRP beam increase uniformly. When the load size is increased to 110kN, the bottom of CFRP plate occurs debonding, the interfacial
bonding stress between FRP and concrete disappears, and the cracks expand rapidly. The deflection size in the mid-span rapidly increase, accompanied with the increase of the load size. In particular, the variation of the deflection is small as well as the load size continuously increases in the initial stage of loading. And the whole loading process, the deflection size of the PHB-CFRP beam at each corresponding stage of loading is less than that of the HB-CFRP beam. The stiffness of PHB-CFRP beam is significantly improved than that of HB-CFRP beam.

When the size of the load is 64kN, the HB-CFRP beam comes out a slight "bang" sound. As the load value is up to 110kN, there is a loud "bang" sound. It is observed that the CFRP plate occurs debonding, not long off, only the left section. While the PHB-CFRP beam has a slight "bang" sound under loading to 140kN. During the process of loading to 200kN, the beam continuously comes out the sound. Finally, when the load value is up to 200kN, there is a loud "bang" sound. It is observed that the CFRP plate occurs debonding (Figure 17) in the symmetrical area near the mid-span. The length and distribution of the debonding of the PHB-CFRP beam are found to be smaller than that of the HB-CFRP beam.

Comparison of the debonding of the anchor section and the non-anchor section of the beams under the condition of high load, it is found that the experimental results are different. The anchor section of the beam does not occur debonding owing to the vertical pressure of the anchorage system, while the non-anchor section obviously occurs debonding. The experimental phenomenon is shown in Figure 18.

In addition, there are two obvious oblique cracks between the load point and the support, and the concrete is crushed under the loading point. The final failure mode belongs to the shear compression failure of the inclined section, which not only is the flexural failure of the normal section, but also is the debonding failure of the interface between concrete and the CFRP plate. Obviously, the strengthening method of PHB-CFRP system can be used to restrain the occurrence of debonding failure, but the shear capacity of the inclined section is insufficient. Otherwise, RC beam strengthened with PHB-CFRP system should have greater flexural load-bearing capacity.
CONCLUSIONS

(1) Comparison and analysis of the flexural performance of the two testing beams, it is concluded that the strengthening method of PHB-CFRP system can obviously improve the flexural capacity of RC beam.
(2) The strengthening method of PHB-CFRP system can restrain the occurrence of cracks, reduce the longitudinal cracking range, and improve the stiffness of RC beam.
(3) The anchorage system improves the cohesive strength of the concrete interface. This strengthening method can effectively restrain the occurrence of interfacial debonding and improve the utilization of CFRP material strength.
(4) For the purpose to improve the flexural performance of RC beam, the shear capacity should be also taken into consideration. So that it can guaranteed that the flexural failure of RC beam is prior to the shear failure.

REFERENCES

PRESENTATION OF THE FASSTBRIDGE PROJECT: FAST AND EFFECTIVE SOLUTION FOR STEEL BRIDGES LIFE-TIME EXTENSION

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ABSTRACT

One of the goals of sustainable development applied to bridge infrastructure is to provide bridge owners with strengthening solutions that may lead to an increase of existing structures service life. In the case of steel bridges, the assessment of the remaining service life is most often linked to the determination of the structural deterioration caused by corrosion and fatigue. Damage caused by fatigue is very difficult to assess before crack initiation, and is more bound to occur in old structures, for which the phenomenon was not taken into account in design before 1970. In addition, old steel materials present a more brittle behavior. The FASSTbridge project, financed through European Infravation call, is aimed at developing a pre-cracking fatigue assessment and strengthening methodology for steel bridges, based on the technology of adhesively bonded composite. The methodology will include an assessment method of remaining fatigue life of existing structures, the design and on-site application of a strengthening system, and the environmental and economical appraisal of the solution. A new strengthening technique based on CFRP plates will be developed. This technique will include the formulation and production of a specific adhesive, specifications for using commercial CFRP plates, and the definition of instrumentation plans to monitor the performance of the strengthening system. Finally an on-site application in an actual steel bridge will be carried out, to verify the quality of the proposed solution.

KEYWORDS

CFRP, Adhesively bonded reinforcement, Steel structures, Fatigue.

INTRODUCTION

Steel bridges serve as vital components in the transportation infrastructure. Bridges are a frequent cause of the major negative impact in densely populated areas with regard to both users’ convenience (such as service disturbances, disruptions, accessibility problems, delays, traffic jams) and welfare (such as obstacles to safety and security, nuisance from noise, vibration dust, and air pollution). Moreover, problems derived from their improper functionality are also cause of important impacts in the economic activities of the affected area (transportation, job accessibility, development strategies). In Europe 15% of the 300 000 bridges are made of steel or they have concrete-steel composite structures (Ye et al. 2014). Of this number, it is considered that approximately 68% need structural interventions. In the USA, 34% of the 599 000 bridges are made of steel. Of this number, approximately 9% are classified as structurally deficient, 15% are functionally obsolete and 9.5% are both structurally deficient and functionally obsolete (Lee 2012). Many of these bridges were constructed using old standards and for a design service life of 50 years, which is coming to its end or has already been exceeded.

Fatigue is the second main cause of damage after corrosion for steel bridges, limiting their load-carrying capacity and residual life, and is one of the main causes involved in fatal mechanical failure of this kind of asset. The increase of traffic flows and loads in the last decades has a direct influence on this issue, especially on structures designed and erected many years ago for which fatigue was not taken into account during design (Palmer 2014). Fatigue is a progressive and local weakening process in which structural damage is accumulated due to the continuous and repetitive application of external loads (vehicles and trucks in the case of steel bridges). The process of fatigue consists of three steps: crack initiation, crack propagation and failure. This phenomenon is extraordinarily dangerous and difficult to identify with a conventional structural stress analysis, which might lead
to a misleading result of safety. In addition, there is no mean to measure fatigue damage on site before crack initiation. Damage can only be assessed using either fatigue design loading from the standards, or long-term on-site strain measurement (Kühn et al. 2008). The corresponding methods, that allow the determination of the remaining service life, can be based on the characteristics of the studied asset (deterministic methods) or in existing statistics (probabilistic). The latter require a considerable amount of data, which is still scarce. Therefore, deterministic methods are commonly used.

Currently, the mainstream strategy concerning fatigue has been a reactive strategy. Maintenance or repair indeed has mainly occurred after the appearance of cracks in the structure. With a bridge stock that is inevitably ageing, it is necessary to widely adopt a preventive strategy to enable road administrations to enlarge the service life of steel and composite steel bridges in a cost-effective and sustainable manner to avoid the high economic and environmental costs of following the current strategy in the years to come. The prevention of fatigue is thus a high priority. To provide such a strategy, it is essential to seek an easy-to-apply solution which includes an engineering analysis methodology for assessing the fatigue damage status and the application of fast, cost-effective and sustainable retrofitting strengthening techniques that enable the wide adoption of the preventive approach, avoiding difficult, resource-consuming and costly retrofitting and repair interventions and demolitions of the steel and composite steel bridges stock.

Once fatigue damage has been identified, normally by visual inspection for crack detection, various methods are frequently used to strengthen steel bridges. The most popular among engineers is the attachment of steel plates to the tension flange of the girders (FHWA 2013) (Figure 1). However, this method has several disadvantages: plates are usually bulky, heavy, difficult to fix and prone to corrosion and fatigue. In general, conventional strengthening techniques are labor-intensive and disruptive to traffic flows, thus limiting their application in the wide network of European and American steel bridges. Carbon Fibre Reinforced Polymers (CFRP) composites, although more expensive than steel plates in terms of price per unit/m², have several and relevant advantages that make them suitable and cost effective for steel bridges retrofitting: their application is less time consuming than traditional solutions (i.e. from one month to a few days, therefore less traffic disruptive), have a high strength-to-weight ratio, excellent fatigue properties, low space disruption, high durability and versatility, and are easy to transport, handle and apply (without heavy equipment) (Tavakkolizadeh et al. 2003; Bocciarelli 2009; Deng 2011) (Figure 1). Despite their wide application on indoor concrete structures, where CFRP is progressively replacing the traditional techniques, CFRP has not been broadly used for steel structures yet, especially in bridges. The main technical barrier for the adoption of CFPR is the durability of existing commercial adhesives in outdoor environments, which is still limited for this kind of interventions, where high functional periods with minimum maintenance are expected. When exposed to outdoor environments and fatigue conditions, in order to ensure cost-effective and durable strengthening interventions, high quality adhesives are necessary (Zhao 2014).

These different statements motivated the proposal to the ERA-Net European Research call Infravation of the project called FASSTbridge (FASSt and effective solution for STEel bridges life-time extension). The project is coordinated by the Spanish research centre Tecnalia, and implies the contribution of two expertise and research organisms (IFSTTAR from France, and MPA Stuttgart from Germany), one Italian company specialized in polymer formulation and production Collanti, two engineering offices (LAP-Consult from Germany and Altavista from United States), the Spanish international building company Dragados SA and a public owner: the Community of
Madrid. The project started on November 2015 and has duration of 24 months. This paper will describe the objectives and implementation plan of the project.

OBJECTIVES OF FASSTBRIDGE PROJECT

FASSTbridge aims at drastically reduce the economic and environmental costs of ownership of the steel bridges stock in Europe and the USA by providing a reliable preventive, cost-effective and sustainable solution for steel bridges life-time extension. The preventive nature of the solution is the key to cost-effectiveness and sustainability, since it will allow the timely design and implementation of innovative, competitive CFRP-based strengthening actions that will reduce the overall costs and environmental impact of life-time extension.

This solution will stand on two pillars. The first will be dedicated to the proposal of a FASSTbridge methodology to prevent the evolution of irreversible fatigue derived problems at a pre-cracking scenario. The second will concern the development of a FASSTbridge strengthening system to preventively extend life-time of steel bridges.

**FASSTbridge methodology**

The methodology will address the development and validation of a preventive and easy-to-apply procedure to assess the remaining life of steel bridges. Such a procedure should be in agreement with existing methods presented in (AASHTO 2012; Kühn et al. 2008), and should propose a clear strategy between the classical nominal stress method and the local stress approach. It should also be supported by indicators from non-destructive or monitoring methods.

If insufficient remaining service life is determined, a proposal will then be made regarding the design and the application of adhesively bonded composite reinforcement for fatigue strengthening operations. This will be based on existing guidelines and previous investigations (CNR 2007; DNV 2012; Schnerch et al. 2007; Cadei et al. 2004).

The methodology will also include a strategy regarding the maintenance of the strengthening, and the monitoring to verify the efficiency of the structural reinforcement and to contribute to databases for the assessment of steel bridges in fatigue.

**FASSTbridge strengthening system**

The proposed strengthening system will rely on the development of a specific adhesive for the considered application. The adhesive should be durable and allow a correct force transfer between CFRP plate and steel adherend. It should also have good rheological performance before curing to allow an easy on site application. The obtained mechanical properties of the cured adhesive should be durable and in agreement with existing standards. Associated to this adhesive, a commercial CFRP plate will be chosen according to the requested technical and economical properties needed for the project.

The compatibility of both parts (adhesive and CFRP plate) will be verified through different tests on the assembly and under different environmental conditions relevant with the design codes recommendations (AASHTO 1989; EN1990 2002; EN 1991 2003). The experiments will also check the compatibility of different monitoring devices with the assembly. The led experimental investigations will be followed by the definition of a pre-certification plan for the studied system.

**FASSTbridge solution**

In addition to the assessment tools and strengthening system, the FASSTbridge solution will be completed with a method to evaluate the effectiveness and efficiency of the interventions based on cost-benefit analysis and life cycle analysis.

An on-site application will be carried out at the end of the project to demonstrate the applicability of the complete solution. This will be done on a composite steel/concrete bridge from the community of Madrid. It will include the application of complete methodology (determination of the remaining life-time, design of the reinforcement, cost-benefit and life-cycle analyses, application of the reinforcement and monitoring on site) (Figure 2).
IMPLEMENTATION

To ensure the achievement of the stated objectives a work plan, including 6 work packages, has been defined for the project (Figure 3). The first package is dedicated to the overall management and coordination of the project. The last package aims at ensuring good dissemination of the results. The other four packages are the technical ones and will be thus more deeply described in the following paragraphs.

**WP2: Life-time expectancy for existing steel bridges**

The work package 2 will be dedicated to design the FASSTbridge methodology regarding the assessment of the remaining fatigue life of the structures, the design of the reinforcement and its maintenance. A review of existing methods regarding fatigue will be carried out for both US and European cases (Figure 4). This should lead to a proposal of an assessment method placed between the classical nominal stress method and the local stress approach (Kim et al. 2015). If the assessment proves one or more detail to be prone to fatigue damage, inspection should be carried out to check the existence of cracks.

In the case of existing cracks, calculation methods based on fracture mechanics will be recommended. Adhesively bonded CFRP may be used to slow crack propagation in such a case (Bassetti et al. 2000; Liu et al. 2009; Lepretre...
et al. 2016), which will not be included in the project since the aim of FASSTbridge is to define a preventive solution.

In the case where no cracking occurred, the second part of WP2 will be dedicated to the definition of the design of strengthening, using adhesively bonded CFRP plates and based on existing recommendations (CNR 2007; DNV 2012; Schnurch et al. 2007; Cadei et al. 2004). This will also include a specific review on the topic of monitoring and the definition of a specific strategy for the studied case.

Figure 4 Fatigue classes according to AASHTO on the left and Eurocode on the right

**WP3: Development and selection of the strengthening components**

Work package 3 aims at defining the FASSTbridge reinforcement system, i.e. developing the adhesive, choosing the CFRP plate and assessing their compatibility. The adhesive will be developed by Collanti according to the specifications of the adhesive defined at the beginning of the project and dedicated to obtain durable and reliable performance. The rheological behavior of the adhesive will be assessed as in (Benzarti et al. 2011; Houhou et al. 2014) to check that the developed adhesive meets the requirements in terms of ease of application, mechanical properties, thermal properties (glass transition temperature) and durability.

The technical and economical properties of the requested CFRP plate will then be determined and a commercial product chosen. Its compatibility with the developed adhesive will be assessed using first prototype testing on the assembly.

Figure 5 Experimental results (glass transition temperature and mechanical tensile behavior) regarding the influence of hydrothermal ageing on two commercial epoxy resins (Benzarti et al. 2011)

**WP4: Strengthening system process qualification**

This work package is entitled to validate the adopted strengthening system through an extensive experimental campaign, designed according to the requirements of the system from the literature (Zhao et al. 2007; Linghoff et al. 2009; Dawood et al. 2010; Deng 2011; Kim et al. 2011; Ghafoori et al. 2015; Hesmati et al. 2015). The influence of the execution process and the temperature will be investigated by static tests as the one defined in (Chataigner et al. 2011). Additionally, specific campaigns dedicated to the study of the durability of the
reinforcement process, will also be carried out similarly to campaigns done in (Chataigner et al. 2012) (Figure 6). The experimental campaign will also be designed to assess the possibility to integrate sensing capabilities to the reinforcement in agreement with the defined methodology.

Work package 4 will also lead to the definition of the cost-benefit and the life-cycle analyses. It will describe the proposed adopted methodology for both cases and the requested information, so that both may be carried out.

![Figure 6 Failure modes in shear of a CFRP to steel bonded assembly before and after hydrothermal ageing](Chataigner et al. 2012)

WP5: Real case demonstration of the solution

The last technical package aims to demonstrate the applicability of the whole proposed solution on a real bridge. For this purpose, the community of Madrid has proposed to analyze one of the bridges within their area of responsibility (Figure 7). The chosen bridge is a composite bridge erected in the 1960s. It has suffered a considerable increase of traffic flow that was not considered during design, but does not present damages.

![Figure 7 Photos of the bridge in Madrid for the on-site application](Chataigner et al. 2012)

The FASSTbridge methodology will be applied on this bridge to assess its remaining fatigue life. A design of an adhesively bonded CFRP reinforcement will then be done for the most critical details. This will allow to carry out both cost-benefit and life cycle analyses of the solution in that particular case and to apply the designed reinforcement system and the chosen monitoring strategy on site.

CONCLUSIONS

The FASSTbridge project aims at developing a fast and effective solution for steel bridges fatigue life-time extension (fasstbridge.eu). The project intends to propose:

- a methodology of structural assessment of remaining service life regarding fatigue, and design of a reinforcement based on adhesively bonded CFRP with additional monitoring,
- in addition to a reinforcement process that will be developed. This one should combine a specifically designed adhesive with a chosen commercial CFRP product.

The overall proposed solution will be completed with cost-benefit and life-cycle assessment tools so that the advantage of the proposed procedure may be evaluated. The project includes a specific work package dedicated to
the application of the whole solution on site on a composite steel-concrete bridge in Madrid. To achieve the comprehensive and extensive work proposed in this project, the consortium of the project is composed by public and private research and expertise organisms, consulting engineering firms, a construction company, a public structure owner, an adhesive production company.

ACKNOWLEDGMENTS

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REFERENCES


FLEXURAL FATIGUE PERFORMANCE OF RC BEAMS STRENGTHENED WITH PRESTRESSED NSM CFRP STRIPS

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ABSTRACT

In the present paper, a total of 10 RC beams strengthened with prestressed and non-prestressed NSM CFRP strips were subjected to static and fatigue loading tests and the monotonic behavior and failure modes of these RC beams with different pre-stress levels were investigated. The influence of various factors, including the CFRP prestress level, CFRP bond length, protection of concrete cover, and the fatigue load levels on the fatigue performance of specimens were analyzed. Test results showed that a high prestress level leads to the cover separation at the end of CFRP prematurely and thus reduces the load carrying capacity of the strengthened beam. However, the failure caused by cover separation at CFRP end can be prevented or delayed by increasing the bond length, wrapping the beam at CFRP end with U-shaped CFRP sheets, and decreasing the prestress level. Additionally, the transformation of failure mode generally leads to great change of fatigue life.

KEYWORDS

Near-surface mounted, CFRP strips, prestress, fatigue performance, cover separation; bond length

INTRODUCTION

Carbon fiber reinforced polymers (CFRP) have been widely used for the strengthening of engineering structures, especially RC bridges. However, by using externally bonded (EB) FRPs, the full strength of FRP can not be utilized (e.g. Masoud et al. 2011) and concrete structures are inclined to fail due to the debonding of EB FRP-concrete interface. Conversely, with the technique of strengthening concrete structures with near surface mounted FRP (NSM FRP), the bonding area between FRPs and concrete is increased. Thus, the debonding of FRP-concrete interface is restrained and the bond capacity of FRP-concrete interface is improved. However, the high strength of FRP is still not fully utilized by the technique of NSM FRPs. On this account, recently researchers have begun to investigate the technique of strengthening structures with prestressed NSM FRPs. It was found that, with regard to this technique, the strength of FRP can be fully utilized, the load carrying capacity of structures significantly improved, and the excellent bond between NSM FRP and concrete can provide anchorage for prestressed FRP (e.g. GUO et al. 2011), and thus the cost of mechanical anchorage required by the strengthening technique with prestressed FRP can be saved. Hence, the technique of strengthening RC structures with prestressed NSM FRP has attracted the most attention from researchers both at home and abroad and a number of investigations in this regard have been conducted. Oudah et al. (e.g. Oudah et al,2012a,2012b ).Tested RC beams strengthening with NSM-CFRP strips prestressed to 0, 20, 40, and 60% of the CFRP ultimate tensile strength. The stress range induced in the tension steel is 125 MPa. All beams were subjected to fatigue loading for 3 million cycles and did not fail. Their experimental test results showed that bond degradation was observed in strengthened beams using NSM-CFRP strips prestressed to 0 and 20%, whereas no signs of bond degradation were observed in beams strengthened using NSM-CFRP strips prestressed to 40, and 60%. They thus concluded that prestressing the CFRP strips enhances the bond between NSM FRP and concrete. (e.g. Fernandes et al. 2012) Tested concrete elements strengthened with NSM FRP laminate strips under fatigue loading and found that all specimens did not fail after 2 million cycles. Besides, according to (e.g. Wahab et al,2012a,2012b) concrete beams strengthened with NSM prestressed CFRP rods exhibited two failure modes: Fracture of steel reinforcement under a low fatigue load and debonding of FRP-concrete interface under a high fatigue load (debonding began at the support and then developed toward the loaded point). In contrast, domestic research on the NSM FRP system (prestressed or non-prestressed) under fatigue loading is scarce and knowledge about the fatigue performance and failure modes of concrete elements is much more limited. Yet, the limited existing research has already proved the complexity of the bond behavior of structures strengthened with NSM FRP.
In view of this, the present study studied the bond behavior of 10 specimens strengthened with prestressed NSM FRP under static and fatigue loading and primarily investigated the effect of various factors (such as CFRP pre-stress level, CFRP bond length, protection of the concrete cover at the end of CFRP, and fatigue load level etc.) on the fatigue behavior of these specimens, aiming at providing theoretical foundation for the application of the technique of strengthening concrete elements with NSM FRP.

TEST PROGRAM

In the experimental study, a total of 10 RC beams strengthened with NSM CFRP were tested, which are with a full length of 3500mm, an effective span of 3300mm, and a sectional dimension of 60mm×350mm. Two reinforcing steel bars with a diameter of 16 mm were used in both the tension zone and the compression zone. Steel stirrups of 8-mm nominal diameter were arranged at a 100 mm interval along the whole length of beams. The design strength of the concrete is C40, and both the pure bending section and the shear bending section are 1100mm long. For detailed dimension and steel reinforcement arrangement, please refer to Figure 1. The bond length of CFRP strips varies from 2900mm to 3300mm, with CFRP strips arranged symmetrically. CFRP strips are Aslan 500 series, with a nominal tensile strength of 2068MPa, an elastic modulus of 131Gpa, and a sectional dimension of 16mm×2.0mm. Epoxy resin is Sikadur®-30 resin from Sika Co. Ltd. Loading was applied by pulse fatigue loading machine with a frequency of 2Hz.

![Figure 1 Specimen Dimension and Steel reinforcement Arrangement (Strips)](image)

Two of the ten specimens were strengthened with prestressed CFRP and subjected to static loading, and one was strengthened with non-prestressed CFRP and tested under fatigue loading. The other 7 specimens were strengthened with prestressed CFRP and to fatigue loading. Test parameters include: CFRP pre-stress level, fatigue loading level, anchorage at the end of CFRP, CFRP bond length. For details, please refer to Table 1.

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<th>Lower fatigue limit /kN</th>
<th>Upper fatigue limit /kN</th>
<th>Bond length/cm</th>
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Note: In specimen IDs, “N” stands for NSM double-groove strengthening, “L” and the number after it for bond length (in cm; the bond length is of 290cm if not specifically given), “P” and the number after it for pre-stress level, “S” for static loading, “F” for fatigue loading and the numbers before and after it for the lower and upper fatigue limits (kN), and “AN” for U-shaped carbon fibre anchorage for the concrete cover at the end of CFRP.
TEST PROCESS AND PHENOMENA

As mentioned above, the present study subjected 10 RC specimens strengthened with NSM FRP strips to static or fatigue loading tests. Detailed testing process and phenomena of each specimen are described as follows:

Specimen NP720-S was strengthened with NSM-CFRP strips prestressed to 720Mpa and subjected to monotonic static test, with a cracking load of 43.2kN. At a load level of 115kN, diagonal cracks were found at the end of FRP, and at 127kN, horizontal branches of these diagonal cracks were observed. At 148kN, the yielding of longitudinal steel reinforcements occurred, at 190kN, the midspan deflection reached 42.9mm, and the concrete cover at the end of CFRP delaminated, the main bar was exposed, and crushed concrete was seen peeling off, making the FRP strip exposed to the air. The whole process is very quick, as shown in Figure 2.

Specimen NP720-19F112 was subjected to fatigue test, with the upper and lower fatigue limits being 60% and 10% of the ultimate load of Specimen NP720-S (112kN and 19kN respectively). When the initial static load was increased to the upper fatigue limit, diagonal cracks of 45° were found in the concrete at the end of FRP, extending to the top of the beam. With the increase in fatigue load, horizontal branches of these diagonal cracks were quickly formed at the steel reinforcement-concrete interface, in very quick growth and forming a typical separation of concrete cover. The fatigue life is only 900 cycles, as shown in Figure 3.

Specimen NP1000-S was strengthened with NSM-CFRP strips prestressed to 1000MPa and subjected to monotonic static test, with a cracking load of only 36.4kN. At a load level of 109kN, diagonal cracks were observed at the end of CFRP, and at 125kN, horizontal branches of these diagonal cracks were found. With the increase in the load, more and more horizontal cracks were seen continuously propagating toward the midspan. At 157kN, horizontal cracks become confluent with one other, resulting in the debonding of the bottom CFRP together with 2cm-thick concrete along the concrete cover at the bottom of the precut groove. Yet, the bond between the debonded concrete and epoxy resin (CFRP strip) was intact. The failure mode of the specimen is as shown in Figure 4.

Specimen NP1000-16F80 was subjected to fatigue test, with the lower and upper fatigue limits being 10% and 50% of the ultimate load of Specimen NP1000-S (16kN and 80kN respectively). The cracking load of the specimen is 43kN. When the initial static load was increased to 79kN, a 6cm-long diagonal crack was found at the end of FRP. After 2200 cycles, horizontal branches of the diagonal crack were seen gradually widening and propagating toward the midspan. After 4000 cycles, horizontal cracks began to decrease in height (2cm down into the groove) and propagated backward to the cracked end of FRP. After 5500 cycles, the backward horizontal cracks resulted in the complete separation of the 2cm-thick concrete cover from the end of CFRP to a place below the loaded point. The failure mode is as shown in Figure 5.

Specimen NP1000-16F64 differs from Specimen NP1000-16F80 only in that the upper fatigue limit was decreased to 40% of the ultimate load of Specimen NP1000-S (64kN). The cracking load of the specimen is 39kN. After static loading, no diagonal cracks were found at the end of FRP. Under fatigue loading conditions, though diagonal cracks were observed at the end of FRP after 6,000 cycles, horizontal branches of these diagonal cracks initiated...
only after 62,000 cycles, propagated into 20cm-30cm long cracks after 140,000 cycles, and stabilized for the rest of 1 million cycles, indicating good fatigue properties of the specimen, as shown in Figure 6. After 1 million cycles, the specimen was subjected to static loading test so as to study its post-fatigue monotonic behavior. At a load level of 128kN, two new horizontal cracks were found below the old horizontal cracks and were seen gradually widening and propagating both forward and backward and finally confluent with each other, resulting in the complete separation of the 2cm-thick concrete cover from the end of CFRP to the mid span. Still, the bond between the separated concrete and epoxy resin was intact. The failure load is 146kN, while the deflection is only 13.4mm. Compared with the specimen subjected to static test, the debonding of the specimen subjected to fatigue test is comparatively slow.

The bond length of Specimen NL310P1000-16F80 was set to be 310cm. Under fatigue loading conditions, diagonal cracks were found at the four ends of the two FRP strips after 16,000 cycles, with the angle varying from 55° to 75°. After 21,300 and 22,900 cycles, horizontal branches of the diagonal cracks initiated in planes A and B respectively and were seen growing at nearly the same speed into 36cm-long and 5-7cm-high cracks after 32700 cycles. After that, the cracks propagated forward and backward: on the one hand, the cracks continued to propagate forward to the midspan and became confluent with the diagonal cracks; on the other hand, the cracks gradually decreased in height and propagated backward to the end of CFRP after 37,700 cycles, resulting in the separation of 2cm-5cm thick concrete cover toward the midspan. The length is about 160cm. Failure mode of the specimen is as shown in Figure 7.

Specimen NP480-16F80 differs from Specimen NP1000-16F8 only in that the prestress was decreased to 480MPa. The cracking load of the specimen was up to 43kN. During the whole process of the fatigue test, no diagonal or horizontal cracks were found at the end of FRP, exhibiting fatigue properties of common RC. After 167,000 cycles, it became impossible to increase or decrease the load, crack width nearby the loaded point increased and specimen deformation got worse. Open the crack, and fatigue failure in longitudinal steel reinforcement was found at the loaded point.

The present study designed Specimen N-10F48 (non-prestressed) as such that the upper and lower fatigue limits were set to be 48kN and 10kN respectively (so as to make the initial strain in the midspan CFRP under static
loading be the same as that of Specimen NP1000-16F80). The cracking load of the specimen is only 28kN. During the whole process of the fatigue test, no cracks were found at the end of FRP and no obvious degradation at other locations. After 796,000 cycles, it suddenly became impossible to increase or decrease the load, main crack width increased, separation of concrete cover occurred, and fatigue failure in steel reinforcement was found.

Specimen NL330P1000-16F80 has a bond length extending to the full effective span (330cm), with other parameters being the same as those of Specimen NP1000-16F80. During the whole process of the fatigue test, no diagonal or horizontal cracks were found in concrete at the end of FRP. After 315,000 cycles, it became impossible to increase or decrease the load, crack width at one side of the loaded point increased abruptly. After 345,000 cycles, crack width at the other side of the loaded point also increased greatly. Open the concrete cover by the method of chiseling, two steel reinforcements in this section were found to have failed due to fatigue loading.

**TEST RESULTS AND DISCUSSION**

**Monotonic behavior of strengthened specimens**

Both specimens submitted to static loading used a 290 cm CFRP bond length. Monotonic behavior of specimens with different CFRP pre-stress levels was investigated. Test results are as shown in Table 2.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>FRP Pre-stress Level /MPa</th>
<th>Cracking Load/kN</th>
<th>Yield Load/kN</th>
<th>Ultimate Load/kN</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>NP720-S</td>
<td>720</td>
<td>43.2</td>
<td>148</td>
<td>190</td>
<td>Separation of concrete cover at the end of CFRP</td>
</tr>
<tr>
<td>NP1000-S</td>
<td>1000</td>
<td>36.4</td>
<td>&gt;157</td>
<td>157</td>
<td>Separation of concrete cover at the end of CFRP</td>
</tr>
</tbody>
</table>

Analyses of midspan strain and deflection showed that: compared with non-prestressed Specimen N-10F48, the deflection of prestressed Specimens NP720-S and NP1000-S decreased by 12% and 37%, steel reinforcement strain decreased by 45% and 50%, cracking load increased by 54% and 30% respectively, indicating that the technique of prestressing resulted in improvement of flexural rigidity and performance of specimens.

Test results also showed that, with a smaller prestress, the specimen failed due to separation of concrete cover at the end of CFRP at the steel reinforcement-concrete interface; with a higher prestress, Specimen NP1000-S failed due to the separation of concrete cover at the interface of groove bottom before the yielding of steel reinforcement. Besides, the ultimate load of Specimen NP1000-S decreases by 18% compared with Specimen NP720-S. Possible reasons are that a higher prestress induced a greater interface shear stress in the groove wall and bottom at the end of FRP, and the combination of load shear stress, flexural rigidity of FRP and debonding stress caused by flexural deformation resulted in the premature debonding of groove bottom interface. These results indicate that prestressed NSM FRP improved the load carrying capacity of specimens. Yet with a higher prestress, specimens may fail due to premature separation of concrete cover at the end of FRP or a thin concrete layer. A even higher prestress may cause the interface debonding at the groove bottom of FRP system.

**EFFECT OF BOND LENGTH ON FATIGUE PERFORMANCE**

Bond length is a crucial factor affecting the load carrying capacity of the concrete at the end of FRP and thus is probably responsible for the separation of the concrete. Additionally, the actual bond length is dependent on the strengthening conditions. Therefore, it is of vital importance to investigate the effect of bond length on the behavior of concrete cover at the end of FRP. Taking 1000MPa as the reference prestress level, the present study investigated the fatigue properties of specimens with 290, 310 and 330cm bond length. Table 3 summarizes the test results.

As can be seen from failure modes listed in Table 3, specimens with 290 and 310 cm bond length failed due to the separation of concrete cover at the end of FRP, exhibiting nearly identical crack propagation and concrete cover declamation. The only difference was that the specimen with a longer bond length (310) degraded at a lower speed and has a longer fatigue life. Still the failure of Specimen with 330 bond length was not due to the debonding at the end of FRP but due to the fracture of steel reinforcement, with no diagonal or horizontal cracks found at the end of FRP or nearby during the fatigue test. From the above analyses, it can be concluded that lengthening the bond length improves the load carrying capacity of the concrete cover at the end of FRP, delays or in some cases
prevents the separation of the concrete cover. The failure mode of the specimen transforms from separation of concrete cover to fatigue failure of steel reinforcement.

### Table 3 Fatigue Properties of Specimens with Different Bond Length

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Bond Length /cm</th>
<th>Fatigue Life /10,000 cycles</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>NP1000-16F80</td>
<td>290</td>
<td>0.55</td>
<td>Separation of concrete cover at the end of CFRP</td>
</tr>
<tr>
<td>NL310P1000-16F80</td>
<td>310</td>
<td>3.77</td>
<td>Separation of concrete cover at the end of CFRP</td>
</tr>
<tr>
<td>NL330P1000-16F80</td>
<td>330</td>
<td>31.5</td>
<td>Fatigue fracture of steel reinforcement</td>
</tr>
</tbody>
</table>

### EFFECT OF LOAD LEVEL ON FATIGUE PERFORMANCE

The fatigue behavior of specimens greatly differs from their monotonic behavior. And the failure mode may vary from specimen to specimen subjected to different load levels. The present study explored the effect of load level on fatigue properties of specimens. Table 4 represents the detailed test results.

### Table 4 Fatigue Properties of Specimens Subjected to Different Load Levels

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Upper load limit /kN</th>
<th>Fatigue Life /10,000 cycles</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>NP1000-16F64</td>
<td>64</td>
<td>100+</td>
<td>Specimens did not fail after 1 million cycles.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Separation of concrete cover at the end of CFRP</td>
</tr>
<tr>
<td></td>
<td></td>
<td>146kN</td>
<td>under static loading</td>
</tr>
<tr>
<td>NP1000-16F80</td>
<td>80</td>
<td>0.55</td>
<td>Separation of concrete cover at the end of CFRP</td>
</tr>
</tbody>
</table>

Table 4 showed that, at a load level of 80kN (the upper fatigue limit), Specimen NP1000-16F80 failed due to separation of concrete cover at the end of CFRP (short fatigue life); yet when the upper fatigue limit decreased to 64kN, Specimen NP1000-16F64 did not fail even after 1 million cycles and only a little damage was done to the concrete cover and no further damage was observed after 297,000 cycles (as shown by Figure 6). It can be seen that the fatigue load level has a noticeable effect on damage to the end of CFRP, for the decrease in the load level will lead to a great decrease in the shear stress, and thus greatly delay or prevent the propagation of horizontal cracks in the concrete cover at the end of CFRP. Though the specimen, in the subsequent static loading test, failed due to separation of concrete cover at the end of FRP, the failure load (146kN) was comparatively low compared with that of Specimen NP1000-S (157kN), indicating that the damage to the concrete cover was not significant after 1 million cycles. Since horizontal cracks in the concrete cover stopped growing after 297,000 cycles and the strain in the steel reinforcement remained to be 500με after 100,000 cycles, the specimen may not fail due to fatigue failure or fracture of steel reinforcement (long fatigue life). Conversely, Specimen N-10F48 failed due to fracture of steel reinforcement after 796,000 cycles, with the strain in the steel reinforcement being around 775με. Therefore, it can be concluded that Specimen NP1000-16F64 is more likely to fail due to fatigue failure of steel reinforcement if the fatigue loading test continues, supporting the existing claim that varying the fatigue load level may result in different failure modes of specimens.

### EFFECT OF PRE-STRESS LEVEL ON FATIGUE PERFORMANCE OF STRENGTHENED BEAMS

As previously analyzed, pre-stress level is another important factor affecting the load carrying capacity of the end FRP. The present study subjected specimens with different prestress levels (yet with the same bond length of 290cm) to fatigue loading tests. To be specific, pre-stress levels were set to be 0MPa, 480MPa, and 1000MPa. Table 5 includes the detailed test results.

As to Group I specimens strengthened with non-prestressed and prestressed FRP strips (prestress level of 1000MPa), the initial strain in the midspan steel reinforcements was set to be the same and specimens were subjected to fatigue loading tests. Test results showed that Specimen N-10F48 differs from Specimen NP1000-16F80 in terms of failure development and the final failure mode. Specimen N-10F48 failed due to steel reinforcement fracture, but with the concrete cover at the end of FRP remained intact and free of cracks all the
time. Conversely, Specimen NP1000-16F80 failed due to the quick separation of the concrete cover. As to Group II specimens with different prestress levels but with the same fatigue load level, Specimen NP480-16F80 failed due to steel reinforcement fracture, while Specimen NP1000-16F80 failed due to separation of concrete cover at the end of FRP caused by horizontal cracks. These phenomena proves that prestress level has a significant influence on the failure mode of specimen. Increasing the prestress level leads to the transformation of failure mode from steel reinforcement fracture to separation of the concrete cover at the end of FRP. It can be concluded that the setting of pre-stress level directly determines the shear stress in the concrete cover at the end of FRP and the fatigue stress in the steel reinforcement at the midspan. Altering prestress level directly results in the transfer of the weakest part, the transformation of the failure mode, and the great change in the fatigue life of the specimen. Therefore, there should be one pre-stress limit which can make the specimen not fail due to separation of concrete cover and improve the flexural properties of the specimen to the greatest extent.

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Prestress / MPa</th>
<th>Fatigue Life /10,000 times</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>N-10F48</td>
<td>0</td>
<td>79.6</td>
<td>Fracture of steel reinforcement</td>
</tr>
<tr>
<td>NP1000-16F80</td>
<td>1000</td>
<td>0.55</td>
<td>Separation of concrete cover at the end of CFRP</td>
</tr>
<tr>
<td>II</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NP480-16F80</td>
<td>480</td>
<td>16.7</td>
<td>Fracture of steel reinforcement</td>
</tr>
<tr>
<td>NP1000-16F80</td>
<td>1000</td>
<td>0.55</td>
<td>Separation of concrete cover at the end of CFRP</td>
</tr>
</tbody>
</table>

CONCLUSIONS
This paper presents an experimental study on mechanical behavior of 10 RC specimens strengthened with prestressed NSM FRP under static and fatigue loading conditions. The following conclusions can be highlighted:
1. Under static loading, increasing the prestress level leads to an improvement of the load carrying capacity of specimens strengthened with NSM CFRP strips. But a very high prestress may cause the earlier separation of the concrete cover at the end of FRP and the decrease of the ultimate load. Specimens with different prestress levels exhibit different failure modes: specimens with a lower prestress level fail due to separation of the concrete cover at the end of FRP, while those with a higher prestress level fail due to separation of bottom concrete cover at the end of FRP. This may be attributed to the fact that a higher prestress level results in a greater stress in the groove wall.
2. The main failure mode observed in all RC beams strengthened with prestressed NSM FRP strips under static and fatigue loading is the separation of concrete cover at the end of FRP. Using U-shaped carbon fiber sheets can effectively constrain the deformation and separation of concrete and thus prevent concrete separation from happening. Decreasing load amplitude can significantly check the propagation of cracks in concrete cover at the end of FRP. Besides, for specimens with an upper load limit of 40%, the cracks at the end of FRP stop growing after 297,000 cycles.
3. Altering the prestress level has a significant effect on the fatigue properties of specimens. With a higher prestress level, the failure mode transforms from steel reinforcement fracture to separation of concrete at the end of FRP and the fatigue life of specimens decreases greatly. This is because, at the same load level, a higher prestress level induces a smaller stress in steel reinforcement yet a greater the stress in the concrete at the end of FRP. Hence, the setting of prestress level directly determines the shear stress in the concrete cover at the end of FRP and the fatigue stress in steel reinforcement. Altering prestress level directly results in the transfer of the weakest part of the specimen.
4. A longer bond length provides a gradual decrease in the stress in concrete at the end of FRP and thus a longer fatigue life of the concrete. After the bond length has been increased to a certain value, the failure mode of the specimen transforms from separation of the concrete cover at the end of FRP to fatigue failure in steel reinforcement, which in turn greatly improves the fatigue life of the specimen.

REFERENCES


Mini-symposium on FRP-Concrete Hybrid Structures

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EFFECT OF GLASS TRANSITION TEMPERATURE ON FLEXURAL BEHAVIOR OF GFRP AND UFC COMPOSITE BEAMS

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ABSTRACT

The fiber reinforced polymers (FRP) have features such as high tensile strength, light weight, high corrosion resistance, and high fatigue resistance. Therefore, they can be used for construction of short span bridges. It was found that the resin matrix contained in the FRP materials becomes soften at the glass transition temperature ($T_g$), so that, their mechanical properties will be deteriorated at $T_g$. The $T_g$ of some commercially available FRP materials is around 60°C and 82°C and hence, the flexural behavior of the bridges constructed using the FRP can be affected by hot environmental temperature. In this study, the influence of elevated temperature on the flexural behavior of composite beams consisting of glass FRP (GFRP) and ultra-high strength fiber reinforced concrete (UFC) was investigated. Large-scale beam bending tests were conducted at temperatures between 20°C and 90°C. The experiment results revealed that the flexural capacity and stiffness of the GFRP I-beams are highly influenced by the glass transition temperature of the vinylester resin matrix. It was found that the use of UFC segments significantly improves both the flexural capacity and stiffness of the GFRP I-beams at temperatures between 20°C and 90°C. This is because of the elimination of premature delamination or kink failure of the GFRP I-beam’s compression flange by the UFC slab. The experiment results were compared with fiber model analysis results and both the results were agreed up to the occurrence of UFC segment slip in the GFRP and UFC composite beams.

KEYWORDS

GFRP, ultra-high strength fiber reinforced concrete, composite beam, glass transition temperature, flexural behavior, fiber model analysis.

INTRODUCTION

The fiber reinforced polymers (FRP) are being used for construction of short span bridges because of their advantages compared to the conventional construction materials, such as high corrosion resistance, high tensile strength, lightweight, and high fatigue resistance. Hai et al., 2010 reported that the superior tensile properties of FRP I-beams cannot be effectively utilized because of the premature delamination failure of the compression flange. This problem was overcome by strengthening the compression flange using ultra-high strength fiber reinforced concrete (UFC) slab (Wijayawardane et al., 2014). Use of the UFC slabs has advantages over reinforced concrete slabs such as high durability, high compressive and tensile strength, and lightweight. The UFC slab consists of precast segments and those segments were connected to the glass FRP (GFRP) I-beam top flange using FRP bolts and epoxy adhesive.

Foster and Bisby (2008) reported that the material properties of the FRPs degrade as the temperature exceeds their glass transition temperature ($T_g$). The $T_g$ of some commercially available FRPs are in between 60°C and 82°C (ACI, 2008). An investigation carried out on a full-scale FRP bridge showed that the maximum temperature at the bridge deck during summer would be approximately 60°C (Sirimanna et al., 2011). Therefore, the elevated temperature may deteriorate the material properties of the GFRP and UFC composite beam materials and as a result, the flexural behavior of these beams will be affected.

The objective of this study is to investigate the flexural behavior of the GFRP and UFC (GFRP-UFC) composite beams subjected to elevated temperature. Large-scale four point bending tests were conducted on the GFRP-UFC composite beams at temperatures between 20°C and 90°C.
EXPERIMENT PROGRAM

Material Properties at Elevated Temperature

The GFRP-UFC composite beam consists of GFRP I-beam, UFC slab, FRP bolts, and epoxy adhesive. The influence of elevated temperature on the mechanical properties of these materials were investigated. Tensile and compressive properties of the GFRP flange and GFRP web, compressive properties of the UFC, and shear strength of the FRP bolts and epoxy adhesive at elevated temperature were investigated and the test results are shown in Table 1, Table 2, and Table 3, respectively. The $T_c$ of vinyl ester resin (resin matrix in GFRP), the resin inside the FRP bolts, and the epoxy adhesive were determined by the test method suggested by Japan Industrial Standards K7121 (JIS, 1987) and they were 58°C, 53°C, and 56°C, respectively. As shown in Table 1, the tensile and compressive strengths of the GFRP flange and GFRP web were significantly deteriorated by elevated temperature. And also, the shear strength of the FRP bolts and the epoxy adhesive were decreased significantly as temperature increases (Table 3). However, as shown in Table 2, there was no significant influence of elevated temperature on the compressive strength of the UFC. This happened because the UFC doesn't contain thermally affected material.

### Table 1 Longitudinal tensile and compressive properties of GFRP flange and GFRP web

<table>
<thead>
<tr>
<th>Property</th>
<th>Specimen size (height × width × effective length)</th>
<th>20°C</th>
<th>50°C</th>
<th>70°C</th>
<th>90°C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile strength of flange (N/mm²)</td>
<td>14 × 10 × 200 (mm)</td>
<td>448</td>
<td>367</td>
<td>369</td>
<td>293</td>
</tr>
<tr>
<td>Young’s modulus of flange (kN/mm²)</td>
<td>21.0</td>
<td>19.8</td>
<td>18.2</td>
<td>17.0</td>
<td></td>
</tr>
<tr>
<td>Tensile strength of web (N/mm²)</td>
<td>10 × 9 × 200 (mm)</td>
<td>306</td>
<td>266</td>
<td>207</td>
<td>174</td>
</tr>
<tr>
<td>Young’s modulus of web (kN/mm²)</td>
<td>17.8</td>
<td>15.5</td>
<td>12.5</td>
<td>12.0</td>
<td></td>
</tr>
</tbody>
</table>

### Table 2 Compressive properties of UFC

<table>
<thead>
<tr>
<th>Property</th>
<th>Cylinder size (diameter × height)</th>
<th>20°C</th>
<th>50°C</th>
<th>70°C</th>
<th>90°C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength (N/mm²)</td>
<td>50 × 100 (mm)</td>
<td>178</td>
<td>173</td>
<td>173</td>
<td>171</td>
</tr>
<tr>
<td>Young’s modulus (kN/mm²)</td>
<td>44.8</td>
<td>42.0</td>
<td>42.0</td>
<td>42.0</td>
<td></td>
</tr>
</tbody>
</table>

### Table 3 Shear strength of FRP bolts and epoxy adhesive

<table>
<thead>
<tr>
<th>Property</th>
<th>Specimen details</th>
<th>20°C</th>
<th>60°C</th>
<th>90°C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear strength of FRP bolt (N/mm²)</td>
<td>Double-lap shear test, two 1/16 bolts, length 80 mm</td>
<td>140</td>
<td>133</td>
<td>94</td>
</tr>
<tr>
<td>Shear strength of epoxy adhesive</td>
<td>Double-lap shear test, 100 × 50 × 2 sides (dimensions in mm)</td>
<td>9.6</td>
<td>3.0</td>
<td>1.7</td>
</tr>
</tbody>
</table>

Beam Test at Elevated Temperature

The GFRP I-beams used in this study were manufactured by pulltrusion. The fiber content in the GFRP I-beams and the mechanical properties of the GFRP laminates used in the composite beams have been reported by Wijayawardane et al. (2014). Two types of beams, 1) GFRP I-beams and 2) GFRP-UFC composite beams were tested under four point bending test. The GFRP I-beams were the control specimens. The overall length and height of the GFRP I-beams are 3,500 mm and 250 mm, respectively. The flange is 14 mm in thickness and 95 mm in width and the web is 9 mm in thickness. The size of a precast UFC segments is 300 × 95 × 35 mm and they were attached to the GFRP I-beams using 16 mm diameter FRP bolts and the epoxy adhesive. The cross-sectional details of the GFRP-UFC composite beam are given in Figure 1. The center-to-center spacing of the FRP bolts is 150 mm. The test variables are shown in Table 4 and Figure 2 illustrates the four-point bending setup of a GFRP-UFC composite beam. The flexural and shear spans of the beams were 700 mm and 1,250 mm, respectively. As shown in Figure 2, in all beams, GFRP stiffeners were attached at both sides of the web to prevent web buckling. Except the beams at 20°C (room temperature), all the other beams were gradually heated up to the test temperature and kept constant at the same temperature for an hour before applying the load. Heating of the beams was done inside a heat insulated steel box which was attached with ten electric heaters. Heating of a GFRP-UFC composite beam prior to bending test is shown in Figure 3. During the bending test, the applied load, strain at midspan section, and the deflection at the midspan were recorded.
RESULTS AND DISCUSSION

Figure 4 shows the failure patterns of the control specimens (GFRP I-beams). Figure 4(a) and Figure 4(b) show that the beams G-20 and G-60 failed by delamination of the compression flange near the loading point. In contrast, beam G-90 failed at the midspan by kinking of the compression flange and the web (Figure 4(c)). The main reason for the kink failure in G-90 beam was softening of the vinylester resin due to the glass transition by elevated temperature. The failure patterns of the GFRP-UFC composite beams are shown in Figure 5 and there were two main failure patterns were observed. According to Figure 5(a) and Figure 5(b), both GC-20 and GC-60 beams failed by crushing of the UFC segmental slab at midspan, whereas beam GC-90 failed by shearing of the FRP bolts and epoxy adhesive in the shear span. In beam GC-60, there was a small slip of the UFC segments in the shear span (Figure 5(b)) but no shearing of the FRP bolts observed. As shown in Figure 5(c), there was no UFC segments crushed at the failure of GC-90 beam, but four UFC segments were completely detached from the I-beam’s top flange in the shear span.

Table 4 Test variables of beam test

<table>
<thead>
<tr>
<th>Specimen name</th>
<th>Beam type</th>
<th>Test temperature (°C)</th>
<th>Availability of UFC slab</th>
</tr>
</thead>
<tbody>
<tr>
<td>G-20</td>
<td>GFRP I-beam</td>
<td>20</td>
<td>No</td>
</tr>
<tr>
<td>G-60</td>
<td>GFRP I-beam</td>
<td>60</td>
<td>No</td>
</tr>
<tr>
<td>G-90</td>
<td>GFRP I-beam</td>
<td>90</td>
<td>No</td>
</tr>
<tr>
<td>GC-20</td>
<td>GFRP-UFC beam</td>
<td>20</td>
<td>Yes</td>
</tr>
<tr>
<td>GC-60</td>
<td>GFRP-UFC beam</td>
<td>60</td>
<td>Yes</td>
</tr>
<tr>
<td>GC-90</td>
<td>GFRP-UFC beam</td>
<td>90</td>
<td>Yes</td>
</tr>
</tbody>
</table>
The load and midspan deflection relationship of the control specimens (GFRP I-beams) and the GFRP-UFC composite beams are shown in Figure 6. At all temperatures, the control specimens showed a significantly low flexural capacity and stiffness compared to those of the GFRP-UFC composite beams. This confirms that the installation of the UFC segments to the GFRP I-beam’s compression flange increases the flexural capacity of the GFRP I-beam approximately by 80%, 70%, and 140% at temperatures 20°C, 60°C, and 90°C, respectively. Therefore, use of the UFC is very important and the UFC slab helps to utilize the superior tensile properties of the GFRP. The flexural capacity and the stiffness of both the control specimens and the GFRP-UFC beams decreased...
as temperature increases. However, there was a large decrease in the flexural capacity and stiffness of both beam types at 90°C. This was resulted by the deterioration of the material properties of the GFRP due to glass transition of the vinylester resin. All control specimens and GC-20 beam showed linear load-deflection relationship until failure. But in the beams GC-60 and GC-90, there was a sudden slip of the UFC segments during loading (Figure 6). The reason for this is the failure of the epoxy adhesive between the UFC and the I-beam’s top flange. The glass transition temperature of the epoxy adhesive is 56°C and the shear test results in Table 3 shows that the epoxy adhesive loses its shear capacity by 68% in between 20°C to 60°C.

FIBER MODEL ANALYSIS (FMA)

The flexural behavior of the GFRP-UFC composite beams was analyzed using a simple Fiber Model and the results were compared with the experimental data. All the GFRP-UFC beams were assumed to be behaved under Bernoulli-Euler theory and the temperature throughout the beams was assumed to be constant. In the fiber model, the GFRP-UFC I-beam is divided into a number of horizontal and longitudinal elements and each horizontal element was assigned with the appropriate material properties given in Table 1, Table 2, and Table 3. It was assumed that the material properties of the GFRP flange and GFRP web elements are homogeneous. In the analysis, full interaction between the GFRP I-beam’s top flange and the UFC slab was considered and hence there was no slip between the flange and the UFC slab. A bi-linear stress-strain relationship from the JSCE design code for UFC structures was used to model the UFC (JSCE, 2004). The shear force developed in the shear span at the I-beam’s top flange-UFC slab interface was calculated using the FMA.

The relationship between the load and midspan deflection obtained from the experiment and the analysis is shown in Figure 7. The lines 1 and 3 in Figure 7 represent the vertical load corresponding to the epoxy adhesive shear capacity at 60°C and 90°C, respectively. The line 2 in Figure 7 shows the vertical load corresponding to the FRP bolt shear capacity at 90°C. The FMA well predicted the flexural behavior of beam GC-20. But in beams GC-60 and GC-90, the analysis results were agreed up to the occurrence of UFC segment slip (Figure 7). However, there was a good agreement with the failure loads between the analysis and the experiment. According to the analysis, it was confirmed that the slipping of the UFC segments in beams GC-60 and GC-90 occurred by the failure of the epoxy bond between the top flange and the UFC slab (lines 1 and 3 in Figure 7, respectively). After failure of epoxy adhesive, shear force between the UFC slab and the top flange was mainly taken by the FRP bolts. Therefore, the failure of the epoxy adhesive did not significantly affect the stiffness of the GFRP-UFC composite beams. In FMA, beam GC-90 failed when the vertical load was equal to the shear capacity of the FRP bolts (line 2 in Figure 7). This confirms that the failure of beam GC-90 was due to shearing of the FRP bolts in the shear span, resulted by the glass transition of the FRP bolts at elevated temperature. In the case of GC-60 beam, the FRP bolt shear capacity was not exceeded at beam failure and that resulted crushing of the UFC segments. Therefore, the failure criteria (flexural failure or shear failure) in these GFRP-UFC composite beams is determined by the shear capacity of the FRP bolts at beam temperature.
CONCLUSIONS

This paper presents the influence of elevated temperature on the flexural behavior of the GFRP-UFC composite beams and the main conclusions of the study are summarized below.

1. The flexural capacity and stiffness of the GFRP I-beams are highly influenced by the glass transition temperature of the vinylester resin.

2. Use of the UFC segments significantly improves both the flexural capacity and stiffness of the GFRP I-beams at room and elevated temperatures (less than 90°C). This is because of the elimination of premature delamination or kink failure of the GFRP I-beam by the UFC slab.

3. The failure pattern (flexural failure or shear failure) of these GFRP-UFC composite beams is determined by the shear capacity of the FRP bolts at the temperature of the beam.

4. The proposed simple fiber model analysis method can be used to analyze the flexural behavior of the GFRP and UFC composite beams up to the occurrence of the UFC segment slip.

REFERENCES

THEORETIC FAILURE MODES MAP FOR GFRP SHS COLUMNS UNDER COMPRESSION WITH EXPERIMENTAL VALIDATION

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ABSTRACT

Although pultruded glass fibre reinforced polymer (GFRP) columns under compression have been investigated in the last two decades, results on square hollow sections (SHS) are still limited. In general, three failure modes are categorized: compressive failure, local buckling and global buckling. In this paper, the critical loads for different failure modes of GFRP SHS columns are analysed. Analytical formulations for the boundaries of different failure modes are received, with quantifications of the effects of column slenderness (non-dimensional slenderness, $\lambda$) and plate slenderness (width-thickness ratio, $b/t$). It demonstrates that local buckling will occur on GFRP columns if the $b/t$ is greater than a critical value; compressive failure will occur if $b/t$ is less than the critical value; global buckling will occur if $\lambda$ is greater than 1 and $b/t$ less than a critical value. Furthermore, a failure modes map against $\lambda$ and $b/t$ is formed. Based on this approach, the failure mode of a GFRP column can be determined with knowing its geometric parameters $\lambda$ and $b/t$, and therefore the corresponding load capacity can be determined accordingly. The failure modes map and load capacities of GFRP SHS columns are well validated by experimental results with different $\lambda$ and $b/t$.

KEYWORDS

GFRP columns, width-thickness ratio, local buckling, failure mode, load-carrying capacity.

INTRODUCTION

Pultruded glass fibre reinforced polymer (GFRP) columns under compression have been widely investigated by researchers from 1990s (Zureick and Scott 1997, Barbero 2000, Mottram et al. 2003), particularly on the failure modes and load-carrying capacities of GFRP columns with open section profiles and different slenderness. The failure modes observed are generally categorized as three types: compressive failure, local buckling and global buckling (Hashem and Yuan 2001). For compressive failure it represents the compressive stress reach the material compressive strength and the specimen crushes in the bottom surface (Hashem and Yuan 2000). Local buckling was reported on the flange of pultruded I and box sections under compression (Tomblin and Barbero 1994, Bank et al. 1996). Theoretical analysis on the critical local buckling load of GFRP plates was carried out (Bank and Yin 1996, Qiao et al. 2001, Kollár 2003) based on orthotropic plate theory and different analytical solutions were received for different boundary conditions at edges. Global buckling is found on slender columns (Barbero and Tomblin 1993) and the critical loads are close to Euler equation. It is also noted that for buckling of GFRP columns, the influence of shear deformation should be considered (Roberts 2002) and a modified Euler equation should be applied on the buckling of GFRP columns (Zureick and Scott 1997). Studies (Bai and Keller 2009, Bai et al. 2009) also indicated slender GFRP columns will exhibit shear failure prior to compressive failure due to low elastic modulus and second-order effects.

Different design equations were proposed by researchers (Barbero and DeVivo 1999, Puente et al. 2006) to predict the load-carrying capacities of GFRP columns at different slenderness. Barbero and Tomblin (1994) investigated the interaction between local buckling and global buckling for GFRP columns with intermediate length, and proposed a design equation with an interaction parameter ($c = 0.84$) to fit experimental results. In their design equation, the load reduction factor $\chi$ is related to the non-dimensional slenderness $\lambda$ and mode interaction parameter $c$. This method is used by (Barbero and DeVivo 1999, Lane and Mottram 2002) with different proposals on interaction parameter $c$ and different expression forms (Puente et al. 2006) on reduction factor $\chi$. In these approaches (Barbero and Tomblin 1994, Mottram 2004, Puente et al. 2006), the local buckling load $P_{local}$ is determined theoretically or experimentally based on short columns; and local-global buckling interaction occurs...
when $\lambda$ is between 0.5 and 1.5 (Bank 2007). However, the occurrence of local buckling also depends on the plate slenderness, i.e. the width-thickness ratios $b/t$ of the plates of GFRP members (Pecce and Cosenza 2000). For example, if a short column with low $b/t$ is under compression, it may exhibit compressive failure rather than local buckling. Therefore no local and global buckling interaction will be received on for such intermediate columns with $\lambda$ between 0.5 and 1.5. As a result the effect of $b/t$ should be considered in the design equations of GFRP columns. A recent work (Cardoso et al. 2014) on the compressive strength equation for GFRP SHS columns has defined a relative plate slenderness $\lambda_p$ for GFRP plates and proposed a design equation for GFRP SHS columns. In this equation the reduction factor $\chi$ is a function with column slenderness $\lambda$ and relative plate slenderness $\lambda_p$. This design equation is validated with experimental results on five different SHS sections. A failure modes map with respect to $\lambda$ and $\lambda_p$ for different failure modes are obtained from experimental results.

In this paper, the effects of $\lambda$ and $b/t$ on different failure modes of GFRP SHS columns under compression have been identified through the analytical formulations, forming the theoretical boundaries of these failure modes. It shows local buckling failure will occur if $b/t$ is greater than a critical value which relating to the material properties of GFRP. A failure modes map for GFRP SHS columns under compression with consideration of effects of both $\lambda$ and $b/t$ is formed. Experiments were performed on two square hollow sections with different $b/t$ at different $\lambda$. The load-carrying capacities corresponding to different failure modes resulted from various $\lambda$ and $b/t$ values are formed and validated by experimental results from current study and literatures.

**ANALYTICAL FORMULATION**

**Critical loads for different failure modes**

The critical loads for three types of failure modes can be determined as follows. For compressive failure (crushing), the ultimate load $P_c$ is determined as:

$$ P_c = A \cdot f_c $$ (1)

where $A$ is the section area and $f_c$ is the compressive strength of material.

For global buckling, the critical load $P_{cr}$ refers to Euler equation:

$$ P_{cr} = \frac{\pi^2 E I}{(kL)^2} $$ (2)

where $E$ is the elastic modulus of the material in longitudinal direction; $I$ is the moment of inertia for section; $kL$ is the effective length of column.

For local buckling, the critical load $P_{local}$ is related to the boundary conditions of the GFRP plates at edges. As SHS section are focused in this study, its boundary condition may be considered as RRSS (Rotational Restricted at two unloaded edges, Simple Support at two loaded edges), which is between two critical cases SS (four edges simple supported) and CCSS (clamped on two unloaded edges), according to (Qiao et al. 2001, Shan and Qiao 2008). The local buckling load $N_{cr}$ for an orthotropic plate at two critical boundary conditions SS and CCSS are given in Eqs (3) and (4) respectively:

$$ N_{cr,SS} = \frac{\pi^2}{b} \left[ 2\sqrt{D_{11} D_{22}} + 2(D_{12} + 2D_{66}) \right] $$ (3)

$$ N_{cr,CCSS} = \frac{\pi^2}{b} \left[ \frac{44.9}{\pi^2} \sqrt{D_{11} D_{22}} + \frac{24}{\pi^2} (D_{12} + 2D_{66}) \right] $$ (4)

where $b$ is the width of the GFRP plate; $D_{11}$, $D_{12}$, $D_{22}$, $D_{66}$ represent different flexural stiffnesses of the plate relating to plate thickness $t$ and material properties such as longitudinal modulus $E_1$, transverse modulus $E_2$, shear modulus $G_{12}$, and poisson ratios $\nu_{12}$ and $\nu_{21}$. Therefore the critical local buckling load for a SHS column is determined as:

$$ P_{local} = \frac{A N_{cr}}{b t} $$ (5)

where $b$ and $t$ are the width and thickness of GFRP plate respectively; $N_{cr}$ is the critical local buckling load of a GFRP plate; $A$ is the cross section area.

**Failure modes map for a column subjected to compression**

Non-dimensional slenderness $\lambda$ (Cardoso et al. 2014) can be defined in Eq (6) considering the two possible failure modes (compressive failure and local buckling) for short columns:

$$ \lambda = \sqrt{\frac{\min(P_c-P_{local})}{P_{cr}}} $$ (6)
where $P_c$ is the compressive failure load; $P_{local}$ represents the local buckling load; $P_{cr}$ is the global buckling load. As $\lambda$ is a parameter that relates to the local buckling and global buckling of GFRP columns, and $b/t$ is a parameter that relates to the local buckling and compressive failure, so the critical load equations for GFRP columns under compression can be derived as functions of both $\lambda$ and $b/t$.

Based on the critical loads for three different failure modes in Eqs (1-3), critical values of $\lambda$ and $b/t$ can be determined at balanced failures when 1) $P_c = P_{cr}$; 2) $P_{local} = P_c$; 3) $P_{cr} = P_{local}$.

Table 5 lists the critical values of $\lambda$ and $b/t$ derived from the balanced failure scenarios for a GFRP SHS column with the boundary condition of CCSS. The results show that non-dimensional slenderness $\lambda$ is an important parameter to distinguish compressive failure and global buckling. When $\lambda > 1$, global buckling will occur because the global buckling load is less than compressive failure load, and vice versa. Critical ratio of width to thickness $(b/t)_{critical}$ plays an important role to distinguish compressive failure and local buckling. As can be seen from Eqs. (7) and (8), $(b/t)_{critical}$ can be calculated based on the material properties of GFRP columns. If $b/t$ is greater than $(b/t)_{critical}$, local buckling will occur because the local buckling load is less than compressive failure load, and vice versa. Finally, local buckling will occur prior to global buckling if the $b/t$ is greater than $\lambda(b/t)_{critical}$, and vice versa. Similar formulations can be derived for a GFRP SHS column considering the boundary condition of SSSS.

<table>
<thead>
<tr>
<th>Balanced failure</th>
<th>Formulation</th>
<th>Failure modes</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_c = P_{cr}$</td>
<td>$\lambda = 1$</td>
<td>$\lambda &gt; 1$, global buckling; $\lambda &lt; 1$, compressive failure.</td>
</tr>
<tr>
<td>$P_{local} = P_c$</td>
<td>$(b/t)_{critical} = \frac{4\pi^2}{E} \frac{4E\xi^2}{12(1-\nu^2\xi^2)} + \frac{4(\nu_1\xi\xi^2 + \xi_0\xi)}{6}$</td>
<td>$b/t &gt; (b/t)<em>{critical}$, local buckling; $b/t &lt; (b/t)</em>{critical}$, compressive failure.</td>
</tr>
<tr>
<td>$P_{cr} = P_{local}$</td>
<td>$(b/t) = \lambda \cdot \frac{4\pi^2}{E} \frac{4E\xi^2}{12(1-\nu^2\xi^2)} + \frac{4(\nu_1\xi\xi^2 + \xi_0\xi)}{6}$</td>
<td>$b/t &gt; \lambda \cdot (b/t)<em>{critical}$, local buckling; $b/t &lt; \lambda \cdot (b/t)</em>{critical}$, global buckling.</td>
</tr>
</tbody>
</table>

The results for both SSSS and CCSS boundary conditions are illustrated in Figure 8. In Figure 8, three types of failure modes are located in three regions formed by boundary lines (determined from balanced failure scenarios) for two boundary conditions (SSSS and CCSS). Compressive failure and local buckling are separated by the boundary according to the balanced failure, i.e. $P_{local} = P_c$, in the form of a horizontal line, only relating to critical values of $b/t$. Global buckling and compressive failure are separated by the boundary according to the balanced failure, i.e. $P_c = P_{cr}$, in the form of a vertical line where $\lambda = 1$. The boundary line between local buckling and global buckling, is obtained considering balanced failure when $P_{cr} = P_{local}$. Therefore it is related to both $\lambda$ and $b/t$.

**Figure 8 Failure modes map for a GFRP column**

**Load-carrying capacities**

The load-carrying capacity of GFRP columns under compression is determined by their failure modes. Commonly, load-carrying capacity is given in the form of reduction factor $\chi$. Based on previous analysis on the failure modes map for a GFRP column, it can be drawn as the functions of $\lambda$ and $b/t$ as summarized in Table 6.

**Table 6 Reduction factor $\chi$ for a GFRP SHS column**
<table>
<thead>
<tr>
<th>Slenderness</th>
<th>Width-thickness ratio (b/t)</th>
<th>Failure modes</th>
<th>Reduction factor $\chi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\lambda&lt;1$</td>
<td>$(b/t) &lt; (b/t)_{critical}$</td>
<td>Compressive failure</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>$(b/t) &gt; (b/t)_{critical}$</td>
<td>Local buckling</td>
<td>$(t/b)^2 \cdot (b/t)_{critical}^2$</td>
</tr>
<tr>
<td>$\lambda&gt;1$</td>
<td>$(b/t) &lt; \lambda \cdot (b/t)_{critical}$</td>
<td>Global buckling</td>
<td>$1/\lambda^2$</td>
</tr>
<tr>
<td></td>
<td>$(b/t) &gt; \lambda \cdot (b/t)_{critical}$</td>
<td>Local buckling</td>
<td>$(t/b)^2 \cdot (b/t)_{critical}^2$</td>
</tr>
</tbody>
</table>

Figure 9 Load-carrying capacity of a GFRP column against $\lambda$ and $b/t$

Figure 9 presents the load-carrying capacity of a GFRP column as a function of both $\lambda$ and $b/t$. The relationship reveals that for short and thick columns with small $\lambda$ and $b/t$, compressive failure is the dominant failure mode. The increase in both $\lambda$ and $b/t$ reduces the load-carrying capacities of GFRP columns, but with different effects on $\chi$. For global buckling failure, $\chi$ is only related to $\lambda$; while for local buckling failure, $\chi$ is related to the actual value of $b/t$ and its critical value $(b/t)_{critical}$.

**RESULTS AND DISCUSSIONS**

Experimental results from Cardoso et al. (2014) and own study (Xie et al. 2016) were summarized to validate above failure modes map. In the latter two SHS sections with similar material properties but different $b/t$ (section A = 10.2, section B = 16.7) were used to investigate the effects of $b/t$ on failure modes and load-carrying capacities.

Figure 10 presents the failure modes of the GFRP columns investigated and the corresponding critical values of $\lambda$ and $b/t$.

Figure 11 Load-carrying capacities for sections with different $b/t$ compared with experimental data
In Figure 10, the critical \( b/t \) values at boundary conditions SSSS and CCSS are calculated from Eqs (3), (4) and (7) based on the GFRP materials properties, resulting into 10.82 and 14.82 respectively for section A and B used in this study. Then the boundary lines for three different failure modes are calculated according to formulations in Table 5 and plotted in Figure 10. Experimental results with different failure modes are summarized according to their \( \lambda \) and \( b/t \) in hollow points (Cardoso et al. 2014) and solid points (Xie et al. 2016) in this failure modes map. Results reveal that satisfactory agreement are received between theoretical predictions and experimental results. Specimens with \( b/t \) less than the critical value and \( \lambda \) less than 1 exhibited compressive failure; and specimens with \( \lambda \) greater than 1 and \( b/t \) less than the critical value exhibit global buckling. For specimens with \( b/t \) greater than the critical values, local buckling is observed. In the transition regions between boundary conditions SSSS and CCSS and \( \lambda = 1 \), interactions between local buckling and global buckling are observed from experimental results. It demonstrated that using this failure modes map to predict the failure mode of a GFRP column is feasible.

The load-carrying capacities of sections with two different \( b/t \) values and slenderness are shown in Figure 11. Based on the previous analysis in Table 6 and Figure 9, the load-carrying capacities, in the form of reduction factor \( \chi \), is related to \( \lambda \) and \( b/t \) values of GFRP columns. Two load-carrying capacity curves, corresponding different \( b/t \) values of 10.2 for section A and 16.7 for section B respectively, are formed in Figure 11. The curve for section A represents the columns will fail in form of compressive failure when \( \lambda < 1 \) and the critical load is therefore dominated by the compressive failure load \( P_c \). When \( \lambda > 1 \), such the columns with \( b/t \) value of 10.2 will fail through global buckling. However, the curve for section B represents that local buckling may occur when \( \lambda < 1 \). This is because their \( b/t \) value (16.7) is greater than the critical value (14.82), and the load-carrying capacity is therefore dominated by the local buckling load. Again, global buckling will still occur for section B when \( \lambda > 1 \), resulting in a theoretical Euler curve.

Experimental results on critical loads for specimens on section A and B at different slenderness are summarized in Figure 11 using square and circular solid points. Results show overall consistence with the above discussions, that for section A specimens with \( \lambda = 0.8 \), the reduction factors \( \chi \) is close to 1 and exhibited compressive failure. Previous design curves (Barbero and Tomblin 1994, Puente et al. 2006) without considering of effects of \( b/t \) on compressive failure may not be able to predict the critical loads here; with \( \lambda > 1 \), the reduction factors \( \chi \) is dominated by Euler curve. A slight deviation on \( \chi \) between experimental results and predictive curve is found on specimens with \( \lambda = 1.22 \). The reason may be due to imperfections on columns leading to global buckling at \( \lambda \) around 1. For experimental results on section B, specimens with \( \lambda = 0.8 \) and 0.83 exhibited local buckling failure, resulting \( \chi \) as 0.74 and 0.76 respectively, which is close to the theoretical prediction strength curve for \( b/t = 16.7 \). For specimens with \( \lambda = 1.20 \) and 1.24, interaction between local buckling and global buckling were found in the experiments, and the reduction factors \( \chi \) as 0.79 and 0.76, compared well with the theoretic predictions.

**CONCLUSIONS**

In this paper, the effects of non-dimensional slenderness \( \lambda \) and width-thickness ratio \( b/t \) on the failure modes and load-carrying capacities of GFRP SHS columns are theoretically analysed. Critical values for \( \lambda \) and \( b/t \) are received for GFRP SHS columns considering both boundary conditions SSSS and CCSS. These Critical values obtained from analytical formulations determine the boundaries between three different failure modes. A failure modes map for GFRP SHS columns under compression considering the effects of \( \lambda \) and \( b/t \) is therefore obtained. It shows that local buckling will occur if \( b/t \) is greater than a critical value \( (b/t)_{critical} \); compressive failure will occur if \( b/t \) is less than the critical value \( (b/t)_{critical} \); global buckling will occur if \( \lambda > 1 \) and \( b/t < \lambda (b/t)_{critical} \). Experimental results from two SHS with different \( b/t \) at different \( \lambda \) and literatures are used to validate theoretical predictions on failure modes and load-carrying capacities of GFRP SHS columns. Overall, good agreements are demonstrated for the failure modes and load-carrying capacities between theoretical predictions and experimental results.

**ACKNOWLEDGMENTS**

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**REFERENCES**


EXPERIMENTAL STUDY ON BOND BEHAVIOR BETWEEN CONCRETE AND GFRP PULTRUDED I-SECTION USING PUSH-OUT TEST

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ABSTRACT

This paper presents the experimental results of a push-out test on the bond behavior between glass fiber reinforced polymer (GFRP) I-section and concrete. The specimen is in the form of a concrete rectangular column with the GFRP I-section (I-section) encased in the middle. Four specimens with different configurations were cast and tested. The main parameters involved bond length and the transverse stirrups. The specimens were divided into two groups in accordance to the bond length. The two specimens in Group A had the same bond length of 300 mm, and the transverse steel stirrups were used at one of the specimens. The bond length of the two specimens in Group B was 450 mm, and one of the specimens was also reinforced with stirrups. Push-out was used to conduct this test and all the I-sections were pushed out. The experimental results show that I-sections with longer bond length have high ultimate bond strength. The development of cracks on the concrete is reduced by the stirrups. Nevertheless, the ultimate bond strength of the specimen is not improved when stirrups were used. In addition, a preliminary bond stress-slip model is proposed, and the theoretical results are in close agreement with the experimental results.

KEYWORDS

Bond-slip, GFRP, pultruded, I-section, concrete, push-out.

INTRODUCTION

Considerable studies have been conducted about the application of GFRP pultruded profiles in hybrid structures in recent years (e.g. Kumar et al. 2004; Correia et al. 2007; EI-Hacha and Chen 2012; Kwan and Ramli 2013). Traditionally, enough bond strength is significant to ensure the transfer of load in hybrid structures (e.g. Malvar et al. 2003; Majdi et al. 2014; Barbieri et al. 2016). Nevertheless, the studies about the bond behavior between GFRP pultruded profiles and concrete are so far limited. Most of the studies about the bond behavior of Fiber Reinforced Polymer (FRP) to concrete are related to FRP strips as well as FRP bars (e.g. Teng et al. 2002; Lu et al. 2005; Lu et al. 2006; Baena et al. 2009; Vilanova et al. 2015). For FRP strips and GFRP pultruded profiles, the bond mechanism at the interface between FRP and concrete is different. Usually the epoxy resins are used to bond the FRP strips on the surface of concrete, therefore, the bond force is mainly offered by the adhesion force of the epoxy. In terms of FRP bars, although no epoxy resins are used between the concrete and FRP bars, the size effect cannot be ignored since the FRP pultruded profiles traditionally have a large surface. Therefore, the previous bond-slip theories for FRP strips and FRP bars are not suitable for the GFRP pultruded profiles.

Due to the increasing application of GFRP pultruded profiles in the hybrid structures, there is a pressing need to investigate the bond behavior between GFRP pultruded profiles and concrete. Therefore, in this experimental study, the bond behavior between GFRP pultruded profiles and concrete was investigated preliminarily. The GFRP pultruded profiles used in this research study was I-section, and the tests were conducted using push-out. The Parameters investigated in this study included the bond length and stirrups. Based on the experimental results, the bond stress-slip curves at the loaded end were analyzed. Furthermore, the mechanism of the load transfer along the interface between the I-section and the concrete was discussed. Lastly, a preliminary bond stress-slip constitutive model was proposed to predict the bond stress, and the predictions from this model were in close agreement with the experimental results.

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EXPERIMENTAL PROGRAM

Design of Specimens

A total of four specimens (Figure 1) were fabricated and tested. Four specimens have the same cross-section with a width of 200 mm and a length of 350 mm. The dimension of the I-section was 10 mm in thickness (both in flange and web), 100 mm in width and 200 mm in height. For all the specimens, the I-section was placed at the center of the concrete, and the web of which was parallel with the long side of the cross-section. A part of the I-section (50 mm or 100 mm height) at the loaded end was left outside of concrete for pushing out. A 50 mm height debonding zone was left at the bottom of the I-section for the debonding. This debonding zone was used to avoid the failure of the concrete around the bottom end of the I-section during the test. Table 1 shows the different configurations of the specimens. The label of the specimens consists of two parts. The first part is the letter A or B indicating the name of the group, and the second part is the letter S denoting that the stirrups were used in this specimen.

<table>
<thead>
<tr>
<th>Group</th>
<th>Specimen</th>
<th>Total Height (mm)</th>
<th>Free end (mm)</th>
<th>Bond length (mm)</th>
<th>Stirrups</th>
<th>Longitudinal bars (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group A</td>
<td>A</td>
<td>400</td>
<td>50</td>
<td>300</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>AS</td>
<td>400</td>
<td>50</td>
<td>300</td>
<td>Steel R10 @ 60</td>
<td>Steel 4 R10</td>
</tr>
<tr>
<td>Group B</td>
<td>B</td>
<td>600</td>
<td>100</td>
<td>450</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>BS</td>
<td>600</td>
<td>100</td>
<td>450</td>
<td>Steel R10 @ 60</td>
<td>Steel 4 R10</td>
</tr>
</tbody>
</table>

Four specimens were divided into two groups (Group A and Group B) in accordance with the different bond length. The bond length of two specimens in Group A is 300 mm. Specimen A was composed of I-section and concrete as shown in Figure 1(a). Steel stirrups were used in Specimen AS to investigate the effect of stirrups on the bond strength [Figure 1(b)]. In order to analyze the influence of the bond length, the bond length of two specimens in Group B were extended to 450 mm. Specimen B in Figure 1(c) was composed of concrete and the I-section. Concrete in Specimen BS was confined by stirrups [Figure 1(d)].

Material Properties and Test Setup

For R10 steel bars, five samples were tested in tension based on the AS 1391 (2007), and the average tensile yield strength was 309 MPa. Concrete cylinders (100 mm × 200 mm/diameter × height) were cast to determine the compressive strength of concrete, and the average compressive strength of concrete at 28 days was 31.8 MPa. The I-sections were provided by the Treadwell Group Company (Treadwell 2016), and were manufactured by a pultrusion technology. The material properties of the I-section were determined in the longitudinal direction both in the web and flange. Tensile testing was conducted following ISO 527 (1997) and the dimension of the five coupons was 25 mm × 250 mm. The average tensile strength was 381 MPa at the flange and 353 MPa at the web. The compressive testing was conducted using ASTM: D695 (2002). Five coupons with dimensions of 12.7 mm × 38.1 mm were tested. The average compressive strength of the coupons was 214 MPa at the flange and 234 MPa at the web.

The push-out test was conducted using the 5000 kN testing machine. As shown in Figure 2(a) and Figure 2(b), the specimens were placed vertically on the testing machine. Two steel blocks were placed under the bottom of the specimen. Adequate space under the specimen was left for the slip of the I-section. One steel plate was placed horizontally at the top of the I-section to distribute the load uniformly. The load and displacement data were
recorded by an electronic data-logger connected to a computer every 2 seconds. After all these setups were done, the specimens were loaded by a displacement controlled load with a rate of 0.1 mm/min. When the I-section was pushed out and the load did not increase, the test was terminated.

EXPERIMENTAL RESULTS AND DISCUSSION

Bond Stress-slip Curves

Table 2 shows the experimental results of the push-out test. The ultimate bond load \((P_s)\) and ultimate load \((P_u)\), ultimate bond stress \((\tau_s)\) and the ultimate stress \((\tau_u)\), the definition of which is discussed in the following sections. All the bond stress mentioned in this study is the average bond stress \((\tau)\) and it is computed as:

\[
\tau = \frac{P}{A}
\]

where \(A\) is the bond area of the I-section and \(P\) is the applied load at the loaded end.

![Schematic diagram of test](image1)

![Test setup](image2)

![Bond stress-slip curves](image3)

**Figure 2** Push-out test and bond stress-slip curves

**Table 2 Experimental results**

<table>
<thead>
<tr>
<th>Group</th>
<th>Specimen</th>
<th>Ultimate bond load (P_s) (kN)</th>
<th>Ultimate bond stress (\tau_s) (MPa)</th>
<th>Loaded end slip at (P_s) (S_{st}) (mm)</th>
<th>Ultimate load (P_u) (kN)</th>
<th>Ultimate stress (\tau_u) (MPa)</th>
<th>Loaded end slip at (P_u) (S_{tu}) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group A</td>
<td>A</td>
<td>109.8</td>
<td>0.46</td>
<td>1.09</td>
<td>116.6</td>
<td>0.49</td>
<td>1.77</td>
</tr>
<tr>
<td></td>
<td>A5</td>
<td>72.1</td>
<td>0.30</td>
<td>1.04</td>
<td>99.7</td>
<td>0.42</td>
<td>2.35</td>
</tr>
<tr>
<td>Group B</td>
<td>B</td>
<td>184.6</td>
<td>0.51</td>
<td>1.61</td>
<td>193.5</td>
<td>0.54</td>
<td>2.39</td>
</tr>
<tr>
<td></td>
<td>BS</td>
<td>122.2</td>
<td>0.34</td>
<td>1.40</td>
<td>138.1</td>
<td>0.38</td>
<td>2.66</td>
</tr>
</tbody>
</table>

All the I-sections in the four specimens were pushed out, and the four specimens showed similar bond stress-slip curves as shown in Figure 2(c). The representative bond stress-slip curve is shown in Figure 3(a). In the first stage (O-A), the initial bond stress increased slowly. Afterwards, the bond stress almost linearly increased from Point A to ultimate bond stress \((\tau_s)\) at Point B with a larger slope. After Point B, the bond stress curve experienced a slight decrease to Point C, and then increased again to the ultimate stress \((\tau_u)\) at Point D. A descending branch could be observed after Point D. Finally, the slip showed a stable increase and the I-section was pushed out gradually.

**Slip process of I-section**

The surfaces of GFRP pultruded profiles are smooth, so the influence of the mechanical bearing force on the bond strength can be ignored for the I-section. Only the chemical adhesion force and the friction force were considered for the bond strength between the I-section and the concrete. The slip process of the I-section is analyzed in Figure 3(b). In this study, the interface between the I-section and the concrete was divided into two zones, the bond zone and the slip zone. The interface in the bond zone was intact without slip, so the bond force in the bond zone was mainly dependent on the chemical adhesion force and the friction force. In the slip zone, the chemical adhesion force was degraded which was attributed to the slip at the interface, so only the friction force contributed to the bond force.
When the I-section was loaded in the initial stage (O-A) as shown in Figure 3(b), the entire interface between the concrete and the I-section was the bond zone which provided the bond force to balance the applied load. As the increase of the applied load, the average bond stress was further improved to balance the increase of the load. Afterwards, different deformation between the concrete and the I-section was enlarged and caused a sudden relative slip at the interface. Therefore, a small slip zone occurred at the top end of the specimen (Point A). This relative slip damaged the original interface and caused the disappearance of the chemical adhesive force. Then, only the friction force was left in this slip zone. It was the reason why a fluctuation at Point A occurred as shown in Figure 3(a). Afterwards, the slip zone was increased further with the increase of the average bond stress and the applied load (Stage A-B). When the bond stress reached the ultimate bond stress (Point B), the I-section could not provide larger bond strength. As a result, the equilibrium of the force was destroyed and the original interface was broken totally. The I-section was pushed out at Point B, and a slight drop at the bond stress-slip curves was observed at the same time. Then, the slip zone was extended to the whole interface (Stage B-C).

The sudden slip at Point B caused a coarse interface with a larger friction coefficient. This new interface could provide a larger friction force to balance the applied load. Hence, the applied load increased again from Point C, and the ultimate load (\(P_u\)) was obtained at Point D as shown in Figure 3(a). With the increase of the slip, the interface was smoothed, as a result, the friction coefficient was decreased, and the load and the friction force reached the equilibrium state again. Lastly, the I-section was pushed out gradually.

Although the ultimate stress (\(\tau_u\)) was obtained at Point D, this maximum stress could not reflect the bond behavior based on the above-mentioned analyses. At Point D, the original interface had been broken totally, and the ultimate stress (\(\tau_u\)) was mainly dependent on the friction force, rather than the chemical adhesion force. Moreover, the influence of the randomness of stage B-C on the stress at Point D cannot be predicted. While the stress at Point B was mainly dependent on the chemical adhesion force at the interface, which could reflect the bond behavior more reasonably. Therefore, the bond stress at Point B was considered as the ultimate bond stress (\(\tau_u\)) in this study.

**Effect of Stirrups and Bond Length**

Based on the analyses of the test results, the bond strength was not improved by the application of stirrups. The reason for this may be that the application of stirrups affected the vibration of concrete during the casting, causing a decrease of the bond at the interface. Nevertheless, the development of the cracks was reduced by stirrups at Specimen AS and Specimen BS.

In terms of the bond length, many previous studies have proved that the bond behavior of steel bars or FRP bars was affected by the bond length, for example, Baena et al. (2009), Lin and Zhang (2013) as well as Torre-Casanova et al. (2013). In this experimental study, the influence of the bond length was investigated through the comparison between Group A and Group B. The test results showed that the longer bond length improved the ultimate bond stress (\(\tau_u\)) at the interface between the I-section and the concrete. For example, the ultimate bond stress (\(\tau_u\)) increased from 0.46 MPa in Specimen A to 0.51 MPa in Specimen B due to the increase of the bond length.

**THEORETICAL ANALYSIS**

Several bond stress-slip constitutive models for FRP bars have been proposed in recent years, and a detailed review had been conducted by Lin and Zhang (2014). Among these models, the BPE model proposed by Eligehausen et al. (1982) is the classical model. This model was developed for the bond of steel bars to concrete, and then successfully used for the bond behavior of FRP bars to concrete by Rossetti et al. (1995). The bond stress-slip
curve in this model is divided into different parts based on some parameters, such as the ultimate bond stress ($\tau_s$), the slip corresponding to the peak bond stress ($s_{ts}$) and the parameters $s_1$, $s_2$, $\alpha$ and $\beta$. This bond stress-slip relationship is given as below:

$$\tau = \tau_s \left( \frac{s}{s_{ts}} \right)^{\alpha} \quad (0 < s \leq s_{ts}) \quad (2)$$

$$\tau = \tau_s \left( s_{ts} < s \leq s_1 \right) \quad (3)$$

$$\tau = \tau_s - (\tau_s - \tau_r) \left( \frac{s - s_1}{s_2 - s_1} \right) \quad (s_1 < s \leq s_2) \quad (4)$$

$$\tau = \tau_r = \beta \tau_s \left( s_2 < s \right) \quad (5)$$

Due to the similar material properties for GFRP bars and GFRP I-section, the BPE model is used to investigate the bond behavior of the GFRP I-section to concrete in this study. During the test, the original interface was broken at Point B, and the stage B-C was difficult to predict due to the randomness. Therefore, the model in this study just focuses on the curvilinear ascending branch of the bond stress-slip curves. Using curve fitting on the experimental results, the parameter $\alpha$ in Eq. 2 is determined as 2.5. Therefore, the bond stress-slip relationship in this study is proposed as:

$$\tau = \tau_s \left( \frac{s}{s_{ts}} \right)^{2.5} \quad (0 < s \leq s_{ts}) \quad (6)$$

where $s$ is the slip at the loaded end and $\tau$ is the average bond stress. The comparison between the theoretical model and the test results are presented in Figure 4. The test results of the ultimate bond stress ($\tau_s$) and the loaded end slip ($s_{ts}$) are used in this calculation. A close agreement is observed in the ascending branch for the four specimens.

The prediction of the bond stress in Eq. 6 requires the given ultimate bond stress ($\tau_s$) and the given loaded end slip ($s_{ts}$). For GFRP bars, some empirical equations were proposed to obtain this bond stress ($\tau_s$) and the corresponding slip ($s_{ts}$). Nevertheless, as a preliminary test, the data is not sufficient for an accurate empirical model to predict these two parameters in this study. Therefore, more studies should be conducted to estimate the ultimate bond stress ($\tau_s$) and the loaded end slip ($s_{ts}$).

CONCLUSION

In this paper, the experimental results and the bond stress-slip model on the bond behavior between the GFRP I-section and the concrete were reported firstly. Four specimens with different configurations were tested using push-out. Based on this research study, the following conclusions are drawn:
The bond behavior of the GFRP I-section to concrete is influenced by stirrups and the bond length. The ultimate bond strength cannot be improved by the stirrups in this study. However, the development of cracks on the concrete is delayed by the application of stirrups. The ultimate bond stress is improved by the longer bond length. A theoretical model is proposed to predict the bond stress-slip relationship at the loaded end of the I-section. The results of the proposed model are in close agreement with the experimental results. Nevertheless, this model is based on the given ultimate bond stress ($\tau_u$) and the corresponding loaded end slip ($s_{te}$). As a preliminary study, the ultimate bond stress and the corresponding loaded end slip cannot be predicted in this study, so a method for predicting the ultimate bond stress and the corresponding loaded end slip needs to be established in future.

**Acknowledgement**

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EXPERIMENTAL INVESTIGATION OF GFRP REINFORCED SQUARE CONCRETE COLUMNS UNDER AXIAL AND ECCENTRIC LOADING

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ABSTRACT

The objective of the study presented in this paper is to expand the understanding of the compression behaviour of square concrete columns internally reinforced with Glass Fiber Reinforced Polymer (GFRP) bars. A total of six columns were tested with the test variables being the type of internal reinforcement (steel bars and GFRP bars) and magnitude of load eccentricity. Results from this study showed that GFRP reinforced columns achieved a lower load carrying capacity as compared to the steel reinforced column specimens for both concentric and eccentric loading conditions. The load carrying capacity of the concentrically loaded GFRP reinforced column was 4.8% lower than its steel counterpart; whereas the load carrying capacity of the eccentricity loaded GFRP reinforced specimens were on average 18.5% lower than their steel counterparts. In addition, the eccentrically loaded GFRP columns achieved an increase in load after concrete spalling unlike the steel reinforced columns.

KEYWORDS

Reinforced concrete column, GFRP bar, eccentric loading, ductility.

INTRODUCTION

The use of reinforcement with FRP composite materials have emerged as one of the alternatives to steel reinforcement for concrete structures prone to corrosion issues (ACI 440.1R–15 2015). However, the mechanical behaviour of FRP reinforcement is different from that of steel reinforcement. In general FRP bars have a higher strength-to-weight ratio, but lower Young’s modulus as compared to steel. Furthermore, when subjected to tension, FRP bars do not experience any plastic behaviour before rupture. Also, the compressive strengths of FRP bars are relatively low compared to the tensile strengths and are subjected to significant variations. Therefore, due to the differences in properties, GFRP bars cannot simply replace steel bars (ISIS 2007). The level of understanding of the behaviour of FRP reinforced compression members has not reached a level where design standards are available for such members. Having said this, the current ACI 440.1R – 15 (2015) design guideline mentions to neglect the compressive contribution of FRP reinforcement when used as reinforcement in columns, in compression members, or as compression reinforcement in flexural members.

Most of the findings of studies investigating FRP reinforced concrete columns have been reported based on testing under concentric loading (Afifi et al. 2014; Tobbi et al. 2012; De Luca et al. 2010). However, in reality for columns nominally carrying only axial compression load, bending moments always exist due to unintentional load-eccentricities and construction errors.

Consequently, this study investigates the behaviour of GFRP-reinforced concrete columns subjected to concentric and eccentric loading. A total of six columns were tested and the failure modes and ductility of the columns were reported based on the results of the experimental program.

EXPERIMENTAL PROGRAM

Design of Columns

In this study, six square concrete columns were cast and tested under concentric and eccentric loads. All the columns had a side dimension of 210 mm and a height of 800 mm. The columns were divided into two groups.
Table 1 shows the test matrix. The first group of columns were reinforced longitudinally and transversally with steel bars (Group RS). The second group of specimens were reinforced with GFRP bars (Group RF). For comparison purposes, the longitudinal and transverse reinforcement ratios of the two groups of specimens were similar. Each group consisted of three specimens with one specimen tested concentrically, one tested under 25 mm eccentricity and one tested under 50 mm eccentricity. The columns are identified by the type of internal reinforcement and magnitude of load eccentricity. For example, Column RF-50 is reinforced with GFRP bars and is eccentrically loaded at 50 mm from the centreline.

<table>
<thead>
<tr>
<th>Column</th>
<th>Material</th>
<th>No. of bars</th>
<th>Diameter (mm)</th>
<th>Reinf. Ratio (%)</th>
<th>Material</th>
<th>Diameter (mm)</th>
<th>c/c spacing (mm)</th>
<th>Eccentricity (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RS-0</td>
<td>Steel</td>
<td>4</td>
<td>12</td>
<td>1.03</td>
<td>Steel</td>
<td>10</td>
<td>50</td>
<td>0</td>
</tr>
<tr>
<td>RS-25</td>
<td>Steel</td>
<td>4</td>
<td>12</td>
<td>1.03</td>
<td>Steel</td>
<td>10</td>
<td>25</td>
<td>25</td>
</tr>
<tr>
<td>RS-50</td>
<td>Steel</td>
<td>4</td>
<td>12</td>
<td>1.03</td>
<td>Steel</td>
<td>10</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>RF-0</td>
<td>GFRP</td>
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<td>12.7</td>
<td>1.15</td>
<td>GFRP</td>
<td>9.5</td>
<td>50</td>
<td>0</td>
</tr>
<tr>
<td>RF-25</td>
<td>GFRP</td>
<td>4</td>
<td>12.7</td>
<td>1.15</td>
<td>GFRP</td>
<td>9.5</td>
<td>25</td>
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<tr>
<td>RF-50</td>
<td>GFRP</td>
<td>4</td>
<td>12.7</td>
<td>1.15</td>
<td>GFRP</td>
<td>9.5</td>
<td>50</td>
<td>50</td>
</tr>
</tbody>
</table>

### Material Testing

All the concrete columns were cast on the same day with ready-mixed, normal strength concrete provided by a local supplier. The average compressive cylinder strength of the concrete at 28 days as determined by the test method AS 1012.9-1999 (Standards Australia 1999) was 29.3 MPa. The Group RS columns were longitudinally reinforced with deformed steel N12 bars and transversally reinforced with plane steel R10 bars fabricated into square ties. The tensile properties of the steel bars were determined in accordance with AS 1391-2007 (Standards Australia 2007) and are reported in Table 2. Sand coated #4 GFRP bars with a nominal diameter of 12.7 mm and #3 GFRP square stirrups of 9.5 mm nominal diameter were used as longitudinally and transverse reinforcement, respectively. Four samples of each diameter were tested in accordance with ASTM D7205-06 (2011) to determine the tensile properties of the GFRP bars, as shown in Table 2. The GFRP reinforcement was supplied by Pultrall Inc. (2012) with a sand-coated surface to improve the bond performance between the bars and surrounding concrete. According to Ehsani et al. (1995) the minimum ratio of radius of bend to the stirrup diameter for GFRP bars is three. However, in this study the radius of bend was only 12.7 mm due to manufacturing errors.

### Column Preparation

The formwork used for moulding the concrete columns was made from 17 mm thick plywood. The longitudinal steel and GFRP bars were cut to 760 mm to ensure a clear cover of 20 mm was maintained at the top and bottom of the reinforcement cage. The steel stirrups used in the Group RS columns were fabricated having similar dimensions to that of the GFRP stirrups but varied in the radius of the bends, as shown in Figure 1. To prevent premature failure at the ends of the columns, two layers of CFRP sheets were wrapped around the top and bottom of the columns with a width of 100 mm. To prevent stress concentrations at the sharp edges, a corner radius of 20 mm was applied at the locations of the CFRP sheet wrapping. In addition, for each eccentrically loaded column specimen, two layers of CFRP wrap were applied longitudinally on the tension zone in combination with the two layers wrapped circumferentially to ensure no premature tensile failure occurred at these regions.
INSTRUMENTATION AND COLUMN TESTING

All of the specimens were tested with the Denison 5000 kN compression testing machine until failure at the University of Wollongong, Australia. The top and bottom ends of the columns were capped with high strength plaster to ensure the bearing surfaces were parallel and the load was distributed evenly during testing. The eccentric load was applied to the column specimen by the interaction of two loading heads attached to both ends of the columns, as illustrated in Hadi and Widiarsa (2015) and shown in Figure 2. Two linear variable differential transformers (LVDT’s) were connected directly to the testing machine at both ends to measure the axial displacements. In addition, a laser triangulation was positioned horizontally at mid-height of the eccentrically loaded columns on the tension side to measure the lateral deflections (δ). The LVDTs and laser triangulation were connected to a data logger to record readings on a control computer at a user controlled time interval.

EXPERIMENTAL RESULTS AND DISCUSSION

Failure modes

The failure modes of the concentrically and eccentrically loaded columns are shown in Figure 3. The failure of the steel reinforced column under concentric loading was marked by the buckling of the four longitudinal bars.
However, the failure mode of the GFRP reinforced column under the same loading was brittle in nature and occurred by the explosive rupture of some of the stirrups and the longitudinal bars. In addition, it was seen that one stirrup experienced slippage (at the splice locations) of the overlap regions.

The failure pattern of the steel reinforced eccentrically loaded columns was characterized by the buckling of the longitudinal bars in compression after the spalling of the concrete cover in the compression region. On the other hand, the failure region of the eccentrically loaded GFRP reinforced was at the top of the specimen in compression and was due to the explosive rupture of the GFRP stirrups and longitudinal bars. This failure area away from the instrumented region may have been due to stress concentrations at the transition of the sharp and rounded edges at the top and bottom ends of the columns. The bars in tension did not fail for all columns.

Ductility and Behaviour of Columns

The experimental results of all the columns are summarized in Table 3. The axial and lateral-displacement relationship of the columns is also shown in Figures 4-6. Initially, all the columns experienced similar behaviour, with the ascending region of the load-displacement curve being almost linear up to the beginning of concrete spalling. A decrease in maximum load of 4.8% relative to Column RS-0 was achieved for Column RF-0. However, a decrease in first maximum load of 19.3% and 17.7% relative to Column RS-25 and RS-50, respectively, was achieved for Column RF-25 and RF-50, respectively. It should be noted that the initial slope of the load-displacement curve of Specimen RF-25 was lower than that of the other specimens which could be due to errors in aligning the specimen resulting in load not being applied exactly at 25 mm eccentricity.

The ductility ($\lambda$) of the columns was calculated based on the ratio of the ultimate displacement ($\delta_u$) divided by the yield displacement ($\delta_y$). For the steel reinforced columns, no increase in strength occurred after concrete spalling and the ultimate displacement was taken at 80% of $P_{\text{max}}$. However, the GFRP reinforced columns were able to sustain an increase in load after the sudden concrete spalling and eventually a second peak load ($P_{\text{peak}}$) was achieved. Therefore, for the GFRP reinforced columns the ultimate displacements was taken at either the first fracture load of the GFRP reinforcement ($P_{\text{fracture}}$), at the peak load ($P_{\text{peak}}$) or at 80% of $P_{\text{peak}}$, whichever gave the smallest axial displacement. For the eccentric loaded GFRP specimens, it was safer to define the ultimate displacement at this peak load rather than at 80% of peak load considering the unpredictable and sudden brittle failure of the internal GFRP reinforcement after peak load unlike the ductile failure mode of the steel reinforced columns.

Based on the ductility definition, the ductility of Column RS-0 was substantially greater than the ductility of the Column RF-0. On the other hand, the ductility of the 25 mm eccentrically loaded columns was similar while the ductility of Column RF-50 was higher than that of Column RS-50. Having said this, the eventual failure mechanism of the GFRP reinforced columns was sudden in nature and brittle, whereas the steel reinforced columns did not fail suddenly but continued to displace until the termination of testing.
<table>
<thead>
<tr>
<th>Column</th>
<th>1st Maximum Load Point</th>
<th>Yield Point</th>
<th>2nd Peak Load, Peak Load, Fracture Load</th>
<th>Ultimate Displ.</th>
<th>Ductility ($\delta_u/\delta_y$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Load, $P_{\text{max}}$ (kN)</td>
<td>Displ. $\delta_y$ (mm)</td>
<td>Load, $P_{\text{yield}}$ (kN)</td>
<td>Displ. $\delta_y$ (mm)</td>
<td>Load, $P_{\text{fracture}}$ (kN)</td>
</tr>
<tr>
<td>RS-0</td>
<td>1350</td>
<td>2.87</td>
<td>1122</td>
<td>1.68</td>
<td>-</td>
</tr>
<tr>
<td>RF-0</td>
<td>1285</td>
<td>2.59</td>
<td>1089</td>
<td>1.58</td>
<td>-</td>
</tr>
<tr>
<td>RS-25</td>
<td>995</td>
<td>2.72</td>
<td>904</td>
<td>2.13</td>
<td>-</td>
</tr>
<tr>
<td>RF-25</td>
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<td>3.00</td>
<td>701</td>
<td>2.20</td>
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<td>RS-50</td>
<td>747</td>
<td>2.65</td>
<td>672</td>
<td>2.02</td>
<td>-</td>
</tr>
<tr>
<td>RF-50</td>
<td>615</td>
<td>2.33</td>
<td>558</td>
<td>1.77</td>
<td>626</td>
</tr>
</tbody>
</table>

$^a$ Displacement at the 80% of $P_{\text{max}}$  
$^b$ Displacement at $P_{\text{fracture}}$  
$^c$ Displacement at $P_{\text{peak}}$

Figure 4 Axial load-deflection curves of the concentrically loaded columns

Figure 5 Axial and lateral load-deflection curves of the 25 mm eccentrically loaded columns
CONCLUSIONS

Based on the experimental results of this study, the following conclusions can be drawn: 1. The steel reinforced columns achieved a higher load carrying capacity as compared to the GFRP reinforced columns for all loading conditions; 2. The load carrying capacity Column RF-0 was 4.8% lower than its steel counterpart; whereas the load carrying capacity of Columns RF-25 and RF-50 loaded eccentrically were on average 18.5% lower than their steel counterparts; 3. The eccentrically loaded GFRP columns achieved an increase in load after concrete spalling unlike the steel reinforced columns; and 4. The ductility of the concentrically loaded steel reinforced column was substantially higher than that of the equivalent GFRP reinforced column. However, the ductility of the eccentrically loaded columns was comparable and even higher than that of the equivalent steel reinforced columns. However, the eventual failure of the GFRP reinforced columns was brittle and sudden in nature.

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HYBRID DOUBLE-SKIN TUBULAR COLUMNS WITH A LARGE RUPTURE STRAIN FRP OUTER TUBE: STUB COLUMN TESTS

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ABSTRACT

Hybrid fiber-reinforced polymer (FRP)-concrete-steel double-skin tubular columns (DSTCs) consist of an outer tube made of FRP and an inner tube made of steel, with the space between filled with concrete. A significant amount of research has been conducted on hybrid DSTCs with an outer tube made of glass FRP (GFRP), carbon FRP (CFRP), or aramid FRP (AFRP). One important finding of the existing research is that the ductility of the column depends significantly on the rupture strain of the FRP tube, among other factors. Against this background, this paper presents an experimental study where hybrid DSTCs with a large rupture strain (LRS) FRP tube, namely, polyethylene terephthalate (PET) FRP tube, were tested under axial compression. PET FRP composites have emerged recently as an economical and environmentally friendly material with a rupture strain of over 7%. Results from a total of four DSTC specimens are presented, with the main test variable being the thicknesses of the steel tube. The test results confirmed the ample ductility of the column and suggested that the diameter-to-thickness ratio of the inner steel tube is a more critical parameter in such DSTCs than in DSTCs with a GFRP, AFRP or CFRP outer tube.

KEYWORDS

FRP, steel, concrete, large rupture strain, confinement, buckling.

INTRODUCTION

Hybrid fiber-reinforced polymer (FRP)-concrete-steel double-skin tubular columns (referred to as hybrid DSTCs) are an emerging form of hybrid columns proposed at The Hong Kong Polytechnic University (Teng et al. 2004, 2007). A hybrid DSTC consists of an outer tube made of FRP and an inner tube made of steel, with the space between filled with concrete (Figure 1). In hybrid DSTCs, the three constituent materials (i.e. FRP, steel and concrete) are optimally combined to achieve several advantages including their excellent corrosion resistance and ductility (Teng et al. 2007). A significant number of studies have been conducted on hybrid DSTCs (e.g. Wong et al. 2008; Yu et al. 2010; Yu and Teng 2013; Fanggi and Ozbakkaloglu 2013; Ozbakkaloglu and Louk Fanggi 2014). The existing studies, however, have generally been limited to the use of an outer tube made of glass FRP (GFRP) (e.g. Wong et al. 2008), carbon FRP (CFRP) (Fanggi and Ozbakkaloglu 2013) or aramid FRP (AFRP) (Fanggi and Ozbakkaloglu 2014). GFRP, CFRP and AFRP are referred to collectively as conventional FRPs hereafter in this paper. One important finding of the existing research is that the ductility of the column depends significantly on the rupture strain of the FRP tube, among other factors. Against this background, this paper presents results from stub column tests of four hybrid DSTCs with a large rupture strain (LRS) FRP tube, namely, polyethylene terephthalate (PET) FRP tube. PET FRP composites have emerged recently as an economical and environmentally friendly material with a rupture strain of over 7%. During the test, the buckling of steel tube in hybrid DSTCs, which is expected to occur under large axial deformations, was paid special attention to.

TEST SPECIMENS

The hybrid DSTC specimens included two pairs of specimens; each pair of specimens were nominally identical. The specimens all had a nominal diameter (i.e. the outer diameter of concrete) of 208 mm and a height of 500 mm. The FRP outer tubes were all composed of three plies of PET-FRP, while two types of steel tubes (i.e. Types A and B) were used. Types A and B steel tubes had the same outer diameter of 139.7 mm, but had thicknesses of 3.5 mm and 5.4 mm respectively, leading to two different diameter-to-thickness (D/t) ratios (i.e. 39.9 and 25.9 respectively). For ease of reference, each specimen is given a name, which starts with a letter to indicate the type
of the steel tube (i.e. A or B) together with an Arabic numeral to indicate the number of plies of FRP (i.e. 3); the Roman numeral at the end is used to differentiate two nominally identical specimens.

![Figure 1 Cross-section of hybrid DSTCs with a PET-FRP tube](image)

All specimens were cast in one batch using ready-mix concrete from a local manufacturer. Results from three standard concrete cylinder (150 mm x 300 mm) tests showed that the elastic modulus, compressive strength and compressive strain at peak stress of the concrete were 25.2 GPa, 28.4 MPa and 0.0025 respectively. For each type of steel tube, tensile tests of two steel coupons were conducted. The average values of elastic modulus, yield stress and tensile strength are 193 GPa, 325 MPa and 470 MPa respectively for Type A steel tubes, while are 194 GPa, 270 MPa and 360 MPa for Type B steel tubes. In addition, tensile tests on six coupons were conducted to determine the mechanical properties of the PET-FRP tube and these tests showed that the PET-FRP used in the present study had an average rupture strain of 0.0956 and an average tensile strength of 823.9 MPa based on a nominal thickness of 0.819 mm per ply.

**TEST SET-UP AND INSTRUMENTATION**

For each specimen, four axial strain gauges and four hoop strain gauges with a gauge length of 20 mm were installed on the outer surface of the FRP tube. These strain gauges were evenly distributed around the circumference at the mid-height of the specimen, with one located at the centre of the overlapping zone. In addition, two axial strain gauges with a gauge length of 10 mm were applied at the mid-height of the steel tube for each specimen. For each specimen, two linear variable displacement transducers (LVDTs) placed 180° apart from each other were used to measure the overall axial shortening, while another two LVDTs placed 180° apart from each other were used to measure the axial deformation of the 150 mm mid-height region. The layout of these LVDTs and the test set-up are shown in Figure 2. To monitor the buckling process of the inner steel tube in the DSTC specimens, a portable action camera was installed on the bottom surface of the top loading plate (Figure 2). All the compression tests were conducted at the University of Wollongong using a 500 ton Denison Compression Testing Machine with a displacement control rate of 0.6 mm per minute.

![Figure 2 Test set-up](image)

**TEST RESULTS AND DISCUSSIONS**

**General Behaviour**

All specimens failed by hoop rupture of the FRP tube at or close to the mid-height of the specimens (Figure 3a). The inner steel tube experienced significant buckling (Figure 3b) which led to considerable load reduction during the test; such load reduction, however, could be recovered in the later stage of testing, and at the ultimate state (i.e. FRP rupture) the specimens could take even a higher load than the first peak (Figure 4). This observation is clearly
different from that from tests of DSTC specimens with a conventional FRP tube (Wong et al. 2008; Yu and Teng 2013; Fanggi and Ozbakkaloglu 2014), and is mainly due to the extremely large axial shortening that the steel tube in the PET-FRP DSTC specimens experienced. Another test observation associated with the extremely large axial deformation was the partial debonding and local buckling of the PET-FRP tube at the finishing end of the overlapping zone (Figure 3). Nevertheless, such debonding/buckling only happened within a small region at the very late stage of tests (i.e. close to the ultimate state), and is thus believed to have little effect on the overall behaviour of the specimens.

![Figure 3 Typical failure mode](image)

**Axial Load-Shortening Behaviour**

The axial load-shortening curves of the four specimens are shown in Figure 4, where all the curves are terminated at the point of FRP rupture. The curves of Specimens A3-I, II can generally be divided into three branches (Figure 4): (1) an approximately bilinear ascending branch before the peak load; (2) a gradual descending branch which was caused by inward buckling of the inner steel tube and the associated inward concrete spalling; (3) another ascending branch where the axial load increased approximately linearly with the axial shortening until the final failure by rupture of the FRP tube. It is evident that the curves shown in Figure 4 are significantly different from the axial load-shortening curves of DSTCs with a conventional FRP tube (Wong et al. 2008; Yu and Teng 2013; Fanggi and Ozbakkaloglu 2014) which typically had an approximately bilinear shape (i.e. similar to the first branch of the curves shown in Figure 4). This is not surprising as the rupture of FRP usually occurs before the first peak if conventional FRPs, which have a much smaller rupture strain than PET-FRP, are used to fabricate the outer tube. For Specimens B3-I, II, Figure 4 shows that the curves of may also be divided into three branches in the same way as discussed above for Specimens A3-I, II, but the load decrease in the second branch and the load increase in the third branch were both much less pronounced. The second and third branches of Specimens B3-I, II may thus also be seen as a single branch with an approximately constant load. It is evident that all the hybrid DSTCs possessed extremely good ductility with ultimate shortenings of up to around 16% of their heights (i.e. 500 mm).

![Figure 4 Axial load-shortening curves](image)
Buckling Behaviour of Inner Steel Tube

The inner steel tubes in the DSTC specimens experienced significant local buckling due to the extremely large axial shortening that they were subjected to, as well as the lack of internal support (Figure 3b). The deformed shapes of the inner steel tube at different stages of loading, as recorded by the action camera (see Figure 2), are shown in Figure 5 for Specimen B3-III. The four subfigures in Figure 5 are corresponding to four points on the load-shortening curve shown in Figure 4, respectively. The buckling process of the inner steel tube can be described as follows: (1) no apparent bucking occurred before the first peak (i.e. Point B), as evident from the deformed shape of the steel tube at Point A when the axial load was 98% that of Point B; (2) at Point B, a slight wrinkle was noticed (Figure 5-B); (3) the first wrinkle developed with the decrease of load, and new wrinkles appeared at various circumferential locations close to the mid-height of the tube (the numbers in Figure 5 indicate the order in which the wrinkles occurred); the wrinkles became quite significant at the trough point of the load-shortening curve (i.e. Point C, see Figure 5-C); (5) after Point C, no new wrinkles were formed, but the existing wrinkles kept progressing with an increasing load until the rupture of the FRP tube (see Figures 5-D).

Figure 5. Buckling process of inner steel tube

CONCLUSIONS

This paper has presented results from four stub columns tests on hybrid DSTCs with a large rupture strain FRP tube (i.e. PET-FRP tube). The test results showed that hybrid DSTCs with a PET-FRP tube possess extremely good ductility despite the severe buckling of the inner steel tube. The test result also showed that the behaviour of PET-FRP DSTCs depends significantly on the diameter-to-thickness ratio of the inner steel tube.

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FRP-CONFINED CONCRETE-ENCASED CROSS-SHAPED STEEL COLUMNS: STUB COLUMN TESTS

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ABSTRACT

FRP-confined concrete-encased cross-shaped steel columns (FCCSCs) are a new form of hybrid columns recently developed at the University of Wollongong. An FCCSC consists of a square FRP outer tube, a cross-shaped steel section and concrete filled in between. This sectional configuration ensures that the concrete is very effectively confined despite the square shape of the column, and that the steel section is well protected and constrained by the FRP tube from corrosion and buckling, leading to a column that is highly ductile and corrosion-resistant. In this paper, results from a series of stub column tests are presented to demonstrate some of the expected advantages of the new column form.

KEYWORDS

FRP, steel section, concrete, tubular column, confinement.

INTRODUCTION

In the past two decades, fiber-reinforced polymer (FRP) has become increasingly popular as a confining material for both the strengthening of existing reinforced concrete columns (Teng et al. 2002) and for new construction (Teng et al. 2007; Yu et al. 2016a). As a result, extensive research has been conducted on the behavior of FRP-confined concrete (e.g. Fam and Rizkalla 2001; Teng et al. 2007; Yu et al. 2010). Existing studies have revealed that the FRP confinement is much more effective in circular columns than in square columns (Teng et al. 2002; Yu et al. 2013).

Against this background, a new form of hybrid columns incorporating an FRP confining tube has recently been developed at the University of Wollongong. The new form of columns is termed FRP-confined concrete-encased cross-shaped steel columns (FCCSCs). An FCCSC consists of a square FRP outer tube, a cross-shaped steel section and concrete filled in between (Figure 1). The square FRP tube typically has four rounded corners and contains fibers close to the hoop direction, while the width of the four flanges of the steel section is typically slightly smaller than that of the four flat sides of the FRP tube. In FCCSCs, the concrete is very effectively confined despite the square shape of the column: the cross-shaped steel section consists of two pairs of flanges connected by webs, so its confinement to the lateral expansion of the concrete infill depends on not only the flexural stiffness of the flanges, but also the axial stiffness of the web; this additional confinement is particularly important to the regions that are otherwise not effectively confined by the square FRP tube (i.e. the regions close to the four flat sides). In addition, the cross-shaped steel section serves as ductile longitudinal reinforcement needed for columns, in particular for those that are subjected to comparable loads in two lateral directions. The FRP tube protects the steel section from environment attacks and constrains its possible buckling, so a layer of concrete cover between the FRP tube and the steel flanges is not always needed. Nevertheless, in the cases where a thin steel section is used, such concrete cover may be provided to reduce the thickness of FRP tube needed for ductile response of the column. FCCSCs can be seen as a variation of concrete-filled FRP tubes with an embedded steel I-section (Karimi et al. 2011; Zakaib and Fam 2012; Yu et al. 2016b).

To demonstrate the concept of FCCSCs, results from two FCCSC specimens tested at the University of Wollongong are presented in this paper. For comparison, results of two square FRP-confined plain concrete (FCPC) columns tested in parallel with the FCCSC specimens are also presented.
TEST SPECIMENS

The test specimens included two nominally identical FCCSC specimens and two nominally identical FCPC specimens. The specimens all had a nominal width of 200 mm (width of the concrete core) and a height of 600 mm. The FRP tubes were fabricated via a wet-layup process by wrapping resin-impregnated glass fiber sheets around a foam core, with an overlapping length of 150 mm; the overlapping zone was limited within one side of the tube. The FRP tubes all had rounded corners with a radius of 25 mm, and were all composed of three plies of FRP. The cross-shaped steel sections were fabricated by welding two H-sections together (i.e. one was cut into two T-shaped sections before welding to the other), and their dimensions are also shown in Figure 1. When preparing for the specimens, the prefabricated FRP tubes were used as the mould for casting concrete; for each FCCSC specimen, a cross-shaped steel section was put into the square FRP tube, which was fixed to a wooden frame, before casting concrete.

All specimens were cast in one batch using ready-mix self-compacting concrete from a local manufacturer. Results from standard concrete cylinder (150 mm x 300 mm) tests showed that the compressive strength and compressive strain at peak stress of the concrete were 35.1 MPa and 0.0026 respectively. For the steel sections, tensile tests of steel coupons showed that the elastic modulus, yield stress and tensile strength are 216.6 GPa, 359.8 MPa and 516.5 MPa, respectively. In addition, an axial compression test was conducted on a steel section, which was identical to those in the FCCSC specimens, and the results showed that its ultimate load was 1662.5 kN. For the FRP tubes, tensile tests on FRP coupons showed that the FRP used in the present study had an average elastic modulus of 74.0 GPa based on a nominal thickness of 0.174 mm per ply.

TEST SET-UP AND INSTRUMENTATION

For each specimen, two linear variable displacement transducers (LVDTs) placed 180° apart from each other were used to measure the overall axial shortening, while another two LVDTs placed 180° apart from each other were used to measure the axial deformation of the 150 mm mid-height region. The layout of these LVDTs and the test
set-up are shown in Figure 2. In addition, for each specimen, five (for FCPC specimens) or seven (for FCCSC specimens) hoop strain gauges with a gauge length of 20 mm were used to measure the hoop strain distributions at the mid-height of the specimens. All the compression tests were conducted at the University of Wollongong using a 500 ton Denison Compression Testing Machine with a displacement control rate of 0.6 mm per minute.

**TEST RESULTS AND DISCUSSIONS**

**Failure Modes**

All the specimens, including g FCCSC specimens and FCPC specimens, failed by the rupture of FRP tube under hoop tension, but the location of FRP rupture was somewhat different for the two types of specimens. Figure 3 shows the four specimens after tests. Figures 3a and 3b shows that the rupture of FRP tube in FCCSC specimens was close to the mid-height of the specimens; the FRP rupture was found to be distributed in both the corner regions and the flat sides (Figure 3b). By contrast, the rupture of FRP tube in FCPC specimens occurred within one of the flat sides close to the top of the columns (Figures 3c and 3d).

![Figure 3 Specimens after test: (a) FCCSC-I; (b) FCCSC-II; (c) FCPC-I; (d) FCPC-II.](image)

**Axial Load-Shortening Behavior**

The axial load-shortening curves of the four specimens are shown in Figure 4, where all the curves are terminated at the point of FRP rupture. Figure 4 shows that the curves of the two FCPC specimens both have a descend branch after the peak load, although the load decrease was generally gradual before the rupture of FRP tube. By contrast, the curves of the two FCCSC specimens feature a bilinear shape, consisting of two ascending branches. This difference was due to the existence of a cross-shaped steel section in FCCSC specimens, which not only contributed directly to the axial load capacity of the columns, but also provided additional confinement to the concrete.

![Figure 4 Axial load-shortening curves](image)

The key test results are summarized in Table 1. In this table, \(P_u\) is the ultimate load of the FCPC or FCCSC specimens from the test; \(S_u\) is the ultimate axial shortening of the FCPC or FCCSC specimens from the test, which is the shortening at the rupture of the FRP tube; \(P_c\) is equal to the peak average stress of confined concrete in the FCPC specimens times the net area of the concrete in FCCSC specimens; \(P_s\) is ultimate load from the compression test of steel section only; \((P_c+P_s)\) represents the ultimate load of the specimen if the steel section did not interact
with the concrete and the FRP tube in FCCSC specimens. It is evident from Table 1 that the ultimate loads \( P_u \) of FCCSC specimens are significantly higher than the \( (P_s + P_p) \) value, indicating that the strength of concrete in the FCCSCs were significantly enhanced by the additional confinement from the cross-shaped steel section. It is also evident that the ultimate shortenings of the FCCSC specimens are significantly higher than those of the two FCPC specimens. This is believed to be due to the following two reasons: (1) the steel section provided additional confinement to the lateral expansion of concrete, so that at the same axial strain, the hoop strain of FRP tube was smaller in FCCSC specimens than in FCPC specimens; (2) the existence of steel section reduced non-uniformity of lateral expansion of concrete, so that the average rupture strains of FRP tube was larger in FCCSC specimens than in FCPC specimens.

### Table 1 Key test results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Ultimate load ( P_u ) (kN)</th>
<th>Average ( P_s ) (kN)</th>
<th>Average ( P_p/(P_s + P_p) )</th>
<th>Ultimate shortening ( S_u ) (mm)</th>
<th>Average ( S_s ) (mm)</th>
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</thead>
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<tr>
<td>FCCSC-I</td>
<td>3302.5</td>
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<td>1.22</td>
<td>12.90</td>
<td>12.22</td>
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<tr>
<td>FCCSC-II</td>
<td>3382.8</td>
<td>1224.1</td>
<td>N/A</td>
<td>6.55</td>
<td>6.81</td>
</tr>
<tr>
<td>FCPC-I</td>
<td>1203.8</td>
<td>1213.9</td>
<td>N/A</td>
<td>7.06</td>
<td></td>
</tr>
<tr>
<td>FCPC-II</td>
<td>1203.8</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### CONCLUSIONS

This paper has presented a newly proposed hybrid column with a square FRP confining tube (i.e. FCCSCs). The new column consists of a square FRP outer tube and a cross-shaped steel section, with the space between filled with concrete. This sectional configuration ensures that the concrete is very effectively confined despite the square shape of the column, leading to a column that is highly ductile. This paper has also presented results from axial compression tests on stub columns to demonstrate the concept of FCCSCs. The results confirmed the excellent performance of FCCSCs. Compared with the concrete in FCPCs, the concrete in FCCSCs has a much larger ultimate axial shortening and a larger compressive strength, when the same FRP tube is used.

### ACKNOWLEDGMENTS

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SEISMIC PERFORMANCE OF CONCRETE-FILLED SQUARE STEEL TUBE WITH FRP-CONFINED CONCRETE CORE

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ABSTRACT

In this study, the experimental programs that investigated the mechanical behavior and seismic performance of concrete-filled square steel tube with fiber-reinforced polymer (FRP)-confined concrete core (SCFC) column were conducted. SCFC specimens were fabricated with three parameters of concrete strength grade, FRP tube thickness and steel tube thickness and tested under axial compression, with four parameters of axial compression ratio, concrete strength grade, FRP tube thickness and steel tube thickness and tested under constant axial compression and reversed-cyclic lateral loading. The failure mode, load-displacement relationship were clarified for the specimens. Additionally, the laser scanning technique was used to obtain the transformations of specimens. In the seismic experiments, the hysteretic responses and envelope curve were received, the Energy-dissipation capacity, the loss of stiffness and equivalent viscous damping ratio were discussed. Results show SCFC columns exhibit favorable energy dissipation and ductility, even when the columns were subjected to high axial loads.

KEYWORDS

Seismic performance, moment-bearing capacity, lateral displacement ductility, stiffness, energy-dissipation, laser scanning.

INTRODUCTION

Researchers devoting to promote the mechanical behavior and seismic performance of columns have been undergoing a steady growth due to the increasing demand for better performance and cost reduction. A number of studies have been carried out to investigate the seismic behaviors of different columns, such as several typical columns including the steel reinforced high-strength concrete column (Varma et al. 2002), the concrete-filled FRP tube column (Zhu and Mirmiran 2006), the concrete-filled steel tube column (Sakino et al. 2002), and shaped-steel reinforced concrete column (Parra-Montesinos and Wight 2000).

Feng et al. (2015) proposed a novel column section named SCFC (Figure 1), which combined three materials within one section in order to take the merits of each material to develop a high level of load-bearing capacity, ductility and the residual load bearing capacity. The section of SCFC is composed of concrete-filled square steel tube with FRP-confined concrete core (FCCC).

AXIAL COMPRESSION EXPERIMENTS

Experimental investigation

The experimental investigation of 62 SCFC stub columns (170 mm in width and 500 mm in height) under axial compression was carried out. The typical specimen under axial compression is shown in Figure 2. The specimens
with a single FCCC were loaded under two schemes, monotonic axial compression and cyclic axial compression. The influences of four design parameters on the mechanical behavior of stub columns were examined. The specimens with multiple FCCC’s were loaded under monotonic compression only. Meanwhile, the effects of sectional configuration, thickness, and material strength on the mechanical behavior were considered. Additionally, a three dimensional scanning technique was adopted during experiments to detect the deformation of specimens at several significant loading steps. From the axial compression experiments, the failure process and failure mode were both obtained, which reveal that the SCFC has several significant advantages which differ from traditional elements, including significant post-yield stiffness and post-peak residual load-bearing capacity.

Figure 2 SCFC stub column under axial compression

Failure mode

In general, most SCFC specimens exhibited similar mechanical behavior when subjected to a monotonically axial compression (Figure 3). Initially, no obvious lateral deformation was observed within 0 to 3 mm of axial displacement. After that, small local bucklings on the steel tube surface dilated slightly outwards. During the subsequent loading process, obvious lateral deformations developed and rupture of the inner FRP tube followed, as evidenced by a loud sound. Meanwhile, the square steel tube showed symmetrical dilation, transforming the section to a circular shape. After rupture of the FRP tube a larger lateral deformation was noticed on the steel tube, with the formation of one to three outward foldings. The loading process of the SCFC specimens was terminated manually when the axial displacement reached around 40 mm to 50 mm.

Figure 3 Significant states of SCFC stub column under axial compression loading

Axial load-displacement relationships

Overall, all SCFC specimens exhibited similar mechanical behavior when subjected to monotonically axial compression and cyclic axial compression, respectively. As shown in Figure 4, the red curve shows the specimen under monotonically axial compression, the grey curve shows the cyclic axial compression for the specimens with same configuration, the black dash curve is the envelope curve obtained from the cyclic curve. The envelope curves of all specimens could be obtained by connecting the initial unloading points on the cyclic stress-strain curves within the strain range for cyclic loading. Overall, the envelope curves of all specimens present the similar trend with monotonic curve, which show the monotonically ascending bi-linear segments with a large post-peak deformation and certain residual load bearing capacity.

Figure 4 Axial load-displacement relationships
Laser scanning for large deformation

Considering the large deformation of specimen, the technique of laser scanning was adopted to obtain the deformations of specimens at prescribed states during loading process. The laser scanning technique could quickly and accurately receive the space coordinates of object, which results of measured graphics could be converted to the coordinates database. Therefore, the deformation situations at prescribed states could be obtained and used to analyze the corresponding stress-deformation relationships. In this experiment, the stub columns have large axial and lateral deformations before failure, which is tough to be measured by strain gauges for that the measured region of strain gauge is limited and the valid measurement range of deformation is generally lower than 20000 µe. Therefore, the laser scanning technique was adopted to obtain the deformations of specimens in this test, which contains three significant steps. As shown in Figure 5, the first step is to distribute the coordinate markers on the specimen for deformation tracing, including the griddings and the points. The second step is to scan the specimens at prescribed states during loading process manually; meanwhile, the scanned graphic could be recorded by data logger. The third step is to convert the graphics to space model for quantified analysis.

Figure 5 Laser scanning steps

Figure 6 shows the corresponding relationship between the 7 prescribed states in axial load-displacement curve and scanned three-dimensional model. The 7 prescribed states in curve include the original point, the middle point during elastic period, the yielding point, the middle point during strengthening period, the peak point, the middle point during post-peak ductile period and the loading terminated point. Figure 6 presents the finite element analysis by using the laser scanning results, which gives the deformation situations of steel tube from square to circular and corresponding confinement stress-strain relationships.

Figure 6 Results of laser scanning

AXIAL AND LATERAL COMBINED LOADING EXPERIMENTS

Experimental investigation

In the seismic experimental investigation (Figure 7), 14 SCFC specimens were fabricated with four differing parameters: axial compression ratio, concrete strength, FRP tube thickness, and steel tube thickness and tested under constant axial compression and reversed-cyclic lateral loading. From the seismic experiments, it shows the high level of load-bearing and energy dissipation capacity of SCFC, which fully took the advantages of material strength and their interaction mechanisms.
Figure 7 SCFC columns under constant axial compression and incrementally increasing lateral loadings

Specimens

A total of 14 specimens are fabricated with four parameters and tested under constant axial compression and incrementally increasing lateral deformation cycles. The four parameters include concrete strength grade (C30, C50, C80), FRP tube wall thickness (3 mm, 5 mm, 8 mm), steel tube wall thickness (6 mm, 10 mm) and design axial compression ratio (0.6, 0.8, 1.0) are investigated, yielding results for 14 different SCFC specimens. The levels of constant axial compression were established to simulate the loading conditions of typical heavy building columns, therefore, 60%, 80% and 100% of nominal column eccentric load capacity were chosen as the axial compression ratios, where the eccentric load capacity have not consider the confinement of FRP tube and steel tube. The specimens were designed as cantilever beams. Each specimen was designed to have a cross section of 300 mm outer width of the column and a cantilever height of 1.5 m. Lateral loading cyclic loading was applied to the specimens at a section 0.2 m below the column tip, which resulted in a shear span (h) of 1.3 m. In addition, each column was fixed into a heavily reinforced concrete footing that was 2100 mm in length, 500 mm in width, and 600 mm in height. To avoid the tearing of welding lines, the two surfaces with welding lines are posed parallel to direction of pull and push.

Experimental observation and failure modes

In general, all specimens exhibited similar hysteretic responses under simulated seismic loading as shown in Figure 7. Initially, no obvious lateral deformation was observed when loading axial compression force and lateral force until the drift ratio reached 0.5 to 1.5%. Around this range of drift ratio, local buckling outwards occur slightly on the steel tube surfaces at 50 mm to 150 mm from the top of the RC footing. After that, with loading process, the local bucklings developed to be a symmetrical rounded elephant foot outwards, as shown in Figure 7. During the subsequent loading process, the ruptures of inner FRP propagated, as evidenced by several crack sounds at a range of drift ratio within 2.0 to 4.0% for majority specimens. Meanwhile, the rupture of FRP indicates the failure of specimen therefore the loading process of specimens was terminated manually.

Hysteretic moment (M)-lateral drift (Δ/h) response

The hysteretic responses of hysteretic moment (M)- lateral drift (Δ/h) for typical specimens are shown in Figure 8. Hysteretic moment (M)-lateral drift (Δ/h) curves that were obtained by averaging the measurements from pull and push directions after correcting lateral moments for P-Δ effects (i.e., $M = F \times h + P \times \Delta$), where $F$ is lateral load, h is shear span of the column, $P$ is the imposed axial load and $\Delta$ is lateral displacement. All the specimens showed a flexure dominant behavior with well-rounded hysteresis loops. Figure 8 shows the hysteretic moment-lateral drift responses of typical specimens, which represents the typical specimens filled with the concrete strength grade of C30, C50 and C80, respectively.
Displacement ductility and moment-bearing capacity

As shown in Figure 9, the envelope curves of all columns were given to compare the hysteretic moment-lateral drift responses. All hysteretic moment-lateral drift responses of specimens can be divided into two types of envelope curves, which include the type I of tri-linear and type II of bi-linear envelope curves. The typical representatives of tri-linear envelope curves include the specimens filled with concrete strength grade of C30 and C50, which following a path of elastic period, strengthening period and softening period. The typical representatives of bi-linear envelope curves include the specimens filled with concrete strength grade of C80, which following a path of elastic period and strengthening period. The difference of bi-linear from tri-linear are without softening period, which are resulted from the rupture of FRP at the last loading cycle before the maximum moment capacity dropping.

Energy-dissipation capacity and equivalent viscous damping ratio

This energy-dissipation capacity of each column was determined at each cycle by calculating the area of corresponding lateral load-displacement hysteretic curve, as shown in Figure 10. Further, the loss of stiffness and the increase of damping ratio due to energy-dissipation were analyzed as two effective criteria, as shown in Figure 11. Last, the equivalent viscous damping ratios of the columns against drift ratio were plotted to evaluates the energy-dissipation capacity of SCFC column (Figure 12). Overall, results show that the seismic performance of SCFC column was demonstrated by 14 specimens to be effective in energy-dissipation and delivered favorable ductility under high axial loads, which is adoptable in regions of high seismicity.

CONCLUSIONS

This paper studied the mechanical behavior and seismic performance of hybrid SCFC sections. Results show that the SCFC column is effective in energy-dissipation and delivered favorable ductility under high axial loads, which is adoptable in regions of high seismicity.
ACKNOWLEDGMENTS

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SCATTER IN THE MECHANICAL PROPERTIES OF RECYCLED CONCRETE AGGREGATE COLUMNS CONFINED BY HAND-LAIRED GFRP UNDER AXIAL COMPRESSION

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ABSTRACT

This paper presents a comparative study of the axial load-bearing capacity and stress-strain curves of recycled aggregate concrete (RAC) and natural aggregate concrete (NAC) columns confined by hand laid-up glass fiber reinforced polymer (GFRP). One significant problem in using RAC in a structural role is the unpredictability of its load-bearing capacity, thus this paper presents a large amount of experimental data to analyze the scatter of confined and unconfined concrete columns under axial compression. Results indicate that GFRP confinement reduces the scatter in the stress-strain response of RAC columns, but owing to the unpredictability of the ultimate rupture state of hand laid-up GFRP, it is ineffective at reducing the scatter in both of ultimate strength and strain. Additionally, systematic differences in stress-strain response between NAC and RAC GFRP-confined columns were demonstrated by the test program and are discussed in this paper.

KEYWORDS

FRP, confined concrete, recycled aggregate concrete (RAC), scatter, mechanical properties.

INTRODUCTION

The production and use of recycled aggregate concrete (RAC) has been researched extensively in recent years as the environmental and economic costs of natural aggregate concretes (NAC) become clear. However, despite its smaller environmental impact and potential for superior economic performance (CANZ TR14 2011), the material has yet to see significant application in structural concrete because it demonstrates poorer short- and long-term performance than NAC does in areas such as slump and constructability (Tangchirapat et al. 2013), compressive strength (Zhao et al. 2014), and creep and drying shrinkage (Yang 2011). An additional problem is the considerable scatter observed in the mechanical properties of RAC, particularly in its compressive strength.

There have been numerous attempts to develop a means of rectifying these performance deficits: researchers have included different admixtures in mix designs to control slump and strength (Tangchirapat et al. 2013), used different moisture conditioning regimens in aggregate storage (Mefteh et al. 2013), and employed different crushing and processing procedures to manufacture recycled aggregates (Akbarnezhad et al. 2013). Unfortunately, the more effective of these methods (crushing and admixture addition) also require potentially significant expenditures, calling into question the economics of RAC use.

There is one other avenue of research into improving the performance of RAC, namely research into confined RAC columns. Traditionally there are two materials used in the confinement of concrete, steel and fiber-reinforced polymer (FRP). At present there is some research on the short- and long-term performance of steel-confined RAC, but only very limited research on GFRP-confined RAC (Xiao et al. 2012; Xiao and Yang 2009; Zhao et al. 2014). Past research on steel-confined RAC has shown many beneficial effects, including increased bearing strength (Yang and Han 2006) and reduced drying shrinkage (Yang 2011).
Steel confinement, however, has a number of disadvantages, including issues with corrosion, self-weight (especially during construction, when it must double as a form for concrete and bear all self- and construction loads), and less effective confinement than FRP with fibers wound radially (Teng and Lam 2004). Thus for certain structural uses FRP confinement is preferable to the use of steel tubes, but at present very little data exists regarding its use with RAC. Xiao and Huang (2012) tested only three GFRP-confined specimens, and while Zhao et al. (2014) tested eighteen, this amounted to only two specimens for any given replacement ratio and degree of confinement.

Given the limited test data and the higher scatter usually observed in the mechanical properties of RAC, it is necessary to produce more test data in order to better understand the behavior of GFRP-confined RAC before it can be applied to real-world structural design. The study performed by the authors tested a significant number of NAC and RAC specimens in order to analyze two phenomena: the scatter in load-bearing capacity of GFRP-confined RAC columns, and the differences in behavior between confined columns made of RAC and NAC. This paper presents the experimental program and the resulting data, analyzes the scatter in mechanical properties of both types of concrete columns, and discusses the qualitative differences in their stress-strain behavior and failure modes.

**EXPERIMENTAL PROGRAM**

**Materials and Preparation**

Natural coarse aggregates came from stockpiles maintained by Tsinghua University’s civil engineering department for research purposes, with maximum nominal sizes of 12.5 mm and 25 mm, respectively, blended to meet the grading requirements laid out in ASTM C33-13 (ASTM 2013) for size number 67 coarse aggregate. Recycled aggregates were from a commercial supplier in Beijing, and their size distribution also met the above standard. Hand sorting of a representative sample of recycled aggregate revealed a deleterious materials content of 2-3% by volume (most impurities were of lower density than stone, so a weight percentage would be misleadingly low), primarily ceramics, glass, organic materials, and foams, with small amounts of dirt, plastic, and reinforcing steel. When casting specimens, the recycled aggregate was intentionally not washed or sieved to remove impurities in an attempt to reflect the ordinary treatment of natural coarse aggregates in real engineering practice. Table 1 gives several other properties of the two types of coarse aggregate.

<table>
<thead>
<tr>
<th>Table 1. Key Aggregate Properties</th>
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</thead>
<tbody>
<tr>
<td>Aggregate Type</td>
</tr>
<tr>
<td>Water Content (%)</td>
</tr>
<tr>
<td>Absorptivity (%)</td>
</tr>
<tr>
<td>Specific Gravity</td>
</tr>
</tbody>
</table>

The mix design used was calculated in accordance with the Chinese standard JGJ55-2011 (CABP 2011) “Specification for Mix Proportion Design of Ordinary Concrete” to the specifications for C30 concrete, with a target 28-day cube compressive strength of 30 MPa, which corresponds to a cylinder compressive strength of approximately 24 MPa (Elwell and Fu 1995). Mix proportions are shown in Table 2. The values for water and coarse aggregate are based on the aggregate being in saturated surface-dry (SSD) condition; in the actual mixing, the amounts of both were altered based on the moisture content of the aggregates, which was measured several days before, to maintain a uniform free water-cement ratio. Slump was controlled and maintained at workable levels (in the range of 80-100 mm) through the addition of water reducer.

<table>
<thead>
<tr>
<th>Table 2. Mix Proportions of Concrete</th>
</tr>
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<tbody>
<tr>
<td>Material</td>
</tr>
<tr>
<td>Water</td>
</tr>
<tr>
<td>Cement</td>
</tr>
<tr>
<td>Fly Ash</td>
</tr>
<tr>
<td>Sand</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
</tr>
</tbody>
</table>
The GFRP confinement layer was formed by hand lay-up using a unidirectional sheet of glass fibers with a nominal thickness of 0.169 mm and an area density of 430 g/m². To verify its material properties, tensile coupon tests were performed in accordance with the Chinese standard GB/T 1447-2005 (AQSIQ and SAC 2005). Five single-layer coupons with midspan width 10mm and length 180 mm were cut and bonded to aluminum clamping surfaces, then tested to tensile failure. The tensile coupon tests of FRP showed that the average tensile strength was 320.1 MPa; the ultimate tensile strain was 0.021; and the secant elastic modulus was 18.7 GPa. Note that modulus of elasticity and tensile strength are both based on the specimen thickness. Additionally, the bonding agent used to affix strain gauges was insufficiently durable to obtain reliable rupture strain results, so rupture strain is taken from test data from previous research using same glass fiber and epoxy.

**Specimens Preparation**

The test specimens were concrete cylinders, each 500 mm long with a radius of 152mm. In total, forty were cast and tested, twenty each of NAC and RAC. The test variable was the presence and thickness of the GFRP confining wraps, so for each type of concrete five columns were wrapped with a single layer, five with two layers, and five with three. The remaining five were a control group which was tested without FRP confinement. GFRP was adhered to the columns by wet lay-up using a structural epoxy resin. For all specimens, 100 mm of overlap was provided between the leading and trailing edges of the FRP wrap. To avoid localized failure at the column ends, a two-layer strip of GFRP 60 mm wide was adhered overtop the confinement. Also, the column ends were sanded flat, roughened, and capped with mortar roughly one week before compression testing to ensure a uniform stress distribution.

Specimens were cured at low room temperature (approximately 15 °C) in the lab under dry conditions, but were not demolded until the day that the GFRP confinement was applied. In accordance with common engineering practice in mainland China, the epoxy was applied and confinement affixed by hand with no machine wrapping or tensioning. Both NAC and RAC specimens were tested at an age of between 27 and 32 days. This spacing was necessary due to the number of specimens. Smaller cylindrical specimens used in strength testing were cast from the same batches of concrete used to make the larger columns, and cured for 28 days under the same conditions as the larger columns. They were demolded shortly before testing.

The specimens were designated as follows: the first letter (N or R) refers to natural or recycled aggregate, the second letter (G or U) refers to glass fiber confinement or unconfined, the third number (0-3) refers to the number of layers of GFRP, and the final number (1-5) is the specimen number. For example, NG-1-4 is the fourth of five specimens with natural aggregate and one layer of GFRP confinement. See Table 3 for details.

**Test Apparatus and Instrumentation**

Compression testing was carried out on a hydraulic compression testing machine with a load capacity of 500 t. Loads were applied at a rate of 2-2.5 kN/s (load-controlled) until total load reached roughly 60% of estimated peak load, then at a rate of 0.5 mm/min (displacement-controlled) until specimen failure. A pre-load of 20 kN was applied to each specimen before testing to limit the effects of column end conditions and imperfect load contact on the stress-strain curve and prevent stress concentrations. Each confined column had four hoop strain gauges and two longitudinal strain gauges spaced uniformly around the circumference at mid-height, while concrete
columns had two hoop and two longitudinal strain gauges attached. All columns additionally had four linear variable differential transformers (LVDTs) placed around them.

RESULTS AND DISCUSSIONS

Overall Behavior

Observation of the loading processes of unconfined RAC and NAC specimens showed the cracking and failure processes in RAC columns were much more gradual and drawn-out. Figure 1 shows the typical failure modes of unconfined RAC and NAC specimens after testing. RAC columns display visible cracking but are substantially more intact and cohesive than NAC ones. Generally, NAC columns shattered upon handling after loading, while RAC columns still maintained sufficient cohesion to remain in one piece.

![Figure 1 Typical failure modes of unconfined specimens: (a) RAC and (b) NAC](image)

For GFRP-confined specimens, it was discovered early in testing that, due to the relatively low temperature in the laboratory where the specimens were cured, the epoxy resin used to apply the GFRP confinement had not fully strengthened, and the first several NAC specimens, including specimens NG-1-1, NG-2-1, NG-2-2, NG-3-1, NG-3-2, experienced delamination failure at the GFRP overlap (Figure 2(a)). When this became apparent all remaining samples were recurred at high temperature for 6 hours, and subsequent tests displayed satisfactory failure modes. Specimens which experienced debonding failure had dramatically lower final bearing capacities and axial and radial strains when compared with those in which the GFRP underwent tensile rupture. They are not included in any computations of specimen average strength or the standard deviation thereof, but their stress-strain phenomena are largely consistent with properly cured specimens up to the point where they experience sudden early failure. As such, their stress-strain behaviors are discussed below.

![Figure 2 Typical failure modes of GFRP-confined specimens: (a) Delamination failure at the GFRP overlap, (b) Fiber rupture, (c) Local fiber rupture followed by delamination (rupture point circled in red)](image)
The remaining specimens, both NAC and RAC, failed through tensile rupture of the confining GFRP layers (Figure 2(b)), sometimes followed by delamination after rupture had already occurred locally (Figure 2 (c)). Rupture most commonly occurred in the middle region of the column, away from the overlap. In some cases rupture occurred simultaneously along the length of an element, while others failed more gradually. The latter phenomenon was particularly common in GFRP-confined RAC specimens, which had a generally more gradual and “softer” failure process than the NAC specimens did. Especially, one phenomenon of note was found upon examination of specimens RG-3-1 and RG-3-3 after the testing program was complete; both had failed at a relatively low level of axial stress and radial strain. After observing video of the experiments it was possible to locate the points at which the GFRP confinement had failed, and upon examination it was found that RG-3-1 had a piece of reinforcing steel protruding beyond the surface of the cracked concrete in that region, while RG-3-3 contained shattered glass. Past research offers no indication that deleterious materials in concrete can cause premature FRP rupture, but it is known that a significant amount of the variability in confining FRP rupture strain is due to localized, unmeasured strains in FRP stemming from the non-uniform nature of concrete cracking. (Lam and Teng 2004).

**Stress-Strain Behavior and Scatter**

The stress–strain curves of the six groups of GFRP-confined specimens are shown in Figure 3(a-f), respectively. It is evident that there are some noticeable differences between GFRP-confined NAC and RAC specimens. The most obvious is that stress–strain curves in the confined RAC specimens have a significantly more “rounded” transition between two relatively linear phases when compared with specimens made of NAC, which is broadly in line with the results observed by Zhao, et al. (2014). Also, there is a pronounced “lag” between concrete cracking and the engagement of the confining GFRP in NAC columns with one and two layers of confinement, which is reflected in the decrease in stress seen in the stress-strain curves (Figure 3(b) and 3(d)) immediately following transition before strengthening begins. This phenomenon has been observed in other research involving relatively low confinement stiffness and high concrete strength (Vincent and Ozbakkaloglu 2013), but is totally absent from the stress-strain curves of the confined RAC columns in this research. This absence is likely due in part to their lower strength.

Especially, the curves of five nominally identical GFRP-confined RAC specimens in each group agree well with each other (Fig. 3(a, c, and e)). These stress-strain curves of GFRP-confined RAC columns in each group follow the very similar development process, but stop in different ultimate condition. The confined specimens exhibited little scatter in stress at any given radial strain before ultimate failure. This effect is most pronounced early in loading, particularly when radial strain is below 60% of its ultimate value. To summarize, GFRP confinement does significantly reduce scatter at points along the stress-strain curve, as the confinement of GFRP acts to resist the cracking and expansion behaviors of concrete. While the tensile rupture state of GFRP cannot be predicted reliably and this unreliability contributes to scatter in ultimate strength, which is discussed in the following section.

**Ultimate Condition and Scatter**

The peak axial stresses ($f_{uc}$) of the confined concrete, corresponding axial strains ($\varepsilon_{uc}$) of all the specimens at ultimate condition are summarized in Table 4, including the average value, standard deviation (Std. Dev.), and coefficient of variation (CV). In line with past research into recycled aggregates and confined RAC columns, the RAC specimens were found to have significantly lower bearing strength than the NAC columns using the same mix proportion, regardless of the presence and degree of GFRP confinement. The average ultimate strength of NAC columns in all three confinement groups was higher than that of RAC columns by roughly 20%. Additionally, there was no significant difference in strength variability between RAC and NAC columns within a confinement group, with all the coefficients of variation around 5%. However, it was found that there was significant variability in the ultimate strain ($\varepsilon_{uc}$) of RAC columns confined by GFRP. RAC columns confined by GFRP failed at variable axial strain: the failure strains of RAC columns with one layer had a coefficient of variation of 10.96%, those with two, 6.14%, and those with three, 11.83%. The coefficient of variation of failure strains of confined NAC columns with one layer of GFRP was also 9.41%, and reliable numbers are not available for two- and three-layer confined NAC columns due to the curing issues mentioned above. Most importantly, GFRP applied to RAC columns by hand using a wet lay-up process does not exhibit any significant restraint on variability compared with unconfined
RAC columns; it was found that the ultimate strength and strain of unconfined NAC and RAC columns had similar variability to the ultimate strengths of such columns which were confined by hand lay-up GFRP.

![Figure 3 Stress-strain curves of GFRP-confined specimens](image)

**Table 4. Variability in strengths and strains at ultimate condition**

<table>
<thead>
<tr>
<th>Element Group</th>
<th>Ultimate strength, $f_{cc}$ (MPa)</th>
<th>Ultimate strain, $\varepsilon_{cc}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Std. Dev.</td>
<td>CV</td>
<td>Average Std. Dev.</td>
</tr>
<tr>
<td>RG-1</td>
<td>37.74</td>
<td>1.50</td>
</tr>
<tr>
<td>NG-1</td>
<td>45.70</td>
<td>1.58</td>
</tr>
<tr>
<td>RG-2</td>
<td>55.18</td>
<td>1.39</td>
</tr>
<tr>
<td>NG-2</td>
<td>65.94</td>
<td>N/A</td>
</tr>
<tr>
<td>RG-3</td>
<td>68.48</td>
<td>3.69</td>
</tr>
</tbody>
</table>
CONCLUSIONS

This experimental program investigated the behavior under axial compression of RAC columns confined by GFRP sheathes formed by hand lay-up processes. Through analysis of property scatter among multiple nominally identical elements and analysis and comparison of the stress-strain behavior of RAC and NAC columns both confined and unconfined, the following conclusions were reached:

1. GFRP confinement formed by hand lay-up processes does not significantly reduce scatter in the axial load-bearing capacity of RAC columns because GFRP ultimate rupture strain and state are not predictable when applied this way. Even though there is evidence that GFRP acts to restrain variability in RAC under compression, this action is not sufficient to outweigh the variability in GFRP’s own failure state.

2. GFRP confinement does significantly reduce scatter at points along the stress-strain curve, as it acts to regularize the cracking and expansion behaviors of concrete. This effect is most pronounced early in loading, particularly when radial strain is below 60% of its ultimate value. Since this level of axial strain is generally not reached until loads reach more than 80% of their ultimate value, in practice, this means that for most of the loading process the stress-strain curves of GFRP-confined RAC columns are significantly more consistent than those of RAC columns left unconfined.

3. The stress-strain curves of GFRP-confined RAC columns follow the same development process as GFRP-confined ordinary concrete: two mostly linear phases, the first with a higher modulus of elasticity and the second with a lower one. However, they have a “rounder” transition zone between the two than those of NAC columns, due to their more gradual cracking process.

4. It is apparently possible for high-hardness impurities such as ceramic or glass in recycled aggregates to cause premature rupture of GFRP confinement. This phenomenon was observed in several columns, but it requires further investigation due to their small numbers and accidental nature.

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REFERENCES


EXPERIMENTAL RESEARCH OF CFFT WITH PULTRUDED FRP PROFILES UNDER AXIAL COMPRESSION

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ABSTRACT

A hybrid member consisting of filament-wound fiber reinforced polymer (FRP) tube, pultruded FRP Profiles, and concrete is presented and experimentally studied in this paper. The hybrid member is in the form of four FRP profiles in contact with the inner surface of the FRP tube. The internally encased FRP profiles are intended to increase the performance of the concrete-filled FRP tube (CFFT) members subjected to combined bending and axial loads. Also the hybrid member has excellent corrosion resistance and is easy to construction. More particularly, as absolutely no steel utilized in this member, the environmental friendly sea water and sea sand can be directly used to cast concrete and filled in this member. The compressive behavior of this hybrid member was investigated under axial, eccentric compression and four-point bending. And the results from axial compression experiments on 4 CFFT with m-Profiles specimens and 3 traditional CFFT specimens are presented in this paper. The results show a reduction in the peak load of CFFT member when m-Profiles were internally encased it. However, the failure initialled from damage on m-Profiles provides an evident warning before overall failure of the member, and the progressive failure process makes the member to exhibit a relatively good failure ductility compared to the explosive failure of CFFT. The mechanical and failure mechanism of this hybrid member under axial compression are presented in this paper.

KEYWORDS

CFFT, FRP tube, FRP profiles, axial compression, failure mode.

INTRODUCTION

The concept of using concrete-filled FRP tubes (CFFT) as columns or piles was first introduced 20 years ago (Mirmiran et al. 1998). Since then axial compressive behavior of FRP-confined concrete has received significant attentions, and it is now well known that the confinement of concrete with FRP composites can substantially enhance the compressive strength and ultimate strain of concrete (Lam and Teng 2003; Mirmiran et al. 1998; Ozbakkaloglu et al. 2013; Pessiki et al. 2001). However, most of the columns in actual construction are subjected to combined axial and bending moments due to load eccentricities, construction errors and lateral loads. In fact, a perfect axially loaded column does not exist. Longitudinal reinforcements are necessary when CFFT member is to be used as a column in the construction. In existing studies, Steel bars (Mandal and Fam 2006; Mohamed and Masmoudi 2010), steel I-section (Karimi et al. 2011; Yu et al. 2016), FRP bars (Hadi et al. 2016) and pultruded FRP profiles (Park et al. 2011) were assembled in CFFT to enhance its performance. Especially, the most highly developed application to date is the use of CFFT as pier columns and piles in marine structures. Steel is easy to get rust in marine salt environment. While FRP bars and pultruded FRP profiles have good resistance to salt (Feng et al. 2014), so they are more advantageous when consider using CFFT member in marine structures. Also the pultruded FRP profiles are easy to customized, and using them can avoid reinforcing bar tying and fixing technology. Combining pultruded FRP profiles, winding FRP tubes and concrete to form a hybrid member has been proposed by Park et al. (2011), in which the pultruded PRP profiles with a quadrilateral closed cross section are assembled to a double-wall cylinder and attached the inside of the FRP tube, then with the space filled concrete. A limited studies reported the behavior of this hybrid member under was similar to traditional CFFT member at
the ultimate axial strain level are just around 0.014 (Park et al. 2011). A new type of hybrid member with some improvements was proposed in this paper. Firstly, the pultruded FRP profiles are designed to have an open cross section to increase the mechanical interlock between them and concrete. Secondly, the pultruded FRP profiles were not assembled to a whole cylinder but placed with adequate intervals, considering a through zone is needed for column-beam joint. The performance of this column were systematically tested under axial, eccentric compression and four-point bending. And the results from axial compression experiments are presented in this paper.

EXPERIMENTAL PROGRAM

Specimen Preparation and Fabrication

Six CFFT with m-Profiles specimens and three reference CFFT specimens were prepared. All the specimens had a nominal diameter of 300 mm (diameter of the concrete core) and a height of 800 mm. They were cast with commercial concrete. The details and cross section of the specimens are given in table 1. Each specimen was given a name, in which the number behind a letter ‘T’ indicates the nominal thickness of FRP tube. For specimens with FRP profiles, there are some letters follow the second dash, in which the “FPs” and “sFPs” indicate FRP profiles and slotted FRP profiles, respectively, and the number before them represents the number of m-Profiles (or slotted m-Profiles) internally assembled. As shown in Figure 1 (a) and (b), before casting the m-Profiles were located and temporarily pasted on the FRP tube.

<table>
<thead>
<tr>
<th>Table 1 Specimen Details</th>
<th>Thickness of FRP tube, ( t_r ) (mm)</th>
<th>Number of m-Profiles (slotted m-Profiles)</th>
<th>Cross-section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Name</td>
<td>Nominal</td>
<td>Measured</td>
<td></td>
</tr>
<tr>
<td>C-T4-4FPs</td>
<td>4</td>
<td>3.75</td>
<td></td>
</tr>
<tr>
<td>C-T4-4sFPs</td>
<td>4</td>
<td>3.75</td>
<td></td>
</tr>
<tr>
<td>C-T8-4FPs</td>
<td>8</td>
<td>6.97</td>
<td></td>
</tr>
<tr>
<td>C-T8-4sFPs</td>
<td>8</td>
<td>6.97</td>
<td></td>
</tr>
<tr>
<td>C-T4</td>
<td>4</td>
<td>3.75</td>
<td></td>
</tr>
<tr>
<td>C-T6</td>
<td>6</td>
<td>5.57</td>
<td></td>
</tr>
<tr>
<td>C-T8</td>
<td>8</td>
<td>6.97</td>
<td></td>
</tr>
</tbody>
</table>

Figure 1 Fabrication and preparation of specimens: (a) locating and temporary fixing of m-Profiles, (b) slotted m-Profiles, and (c) casting concrete

Material Properties

The mechanical properties of prefabricated filament-wound glass FRP (GFRP) tubes in hoop direction were tested, using a tensile split-disk tests according to ASTM D2290-12 (ASTM 2012). Five FRP rings, each having a uniform height of 40 mm, were cut from the FRP tube with a thickness of 3.75 mm. Six hoop strain gauges with a gauge
length of 10 mm were installed, among which two were centered at the top and bottom of rings, whereas the other four gauges were located at 15 mm away from the gaps. The readings of the six strain gauges were found to be close. The average elastic modulus ($E_T$) is 43.64 GPa.

Two type of pultruded FRP profiles used in this study were both customized to have a m-shape cross-section with a curved edge having the same radian as the used FRP tube, as shown in the Figure 2. The only difference of them is whether it was conducted by slotting process. The one with the whole three stiffeners was referred as m-Profiles (Figure 2 (a)), the other one was named as slotted m-Profiles with a letter “s” indicating it was reprocessed by slotting on the two stiffeners after pultrusion. The slotting was shaped as a half round with the diameter and the interval are both 30 mm, as shown in Figure 2 (b). It was intense to increase the mechanical interlock between FRP profiles and concrete. 5 couples were cut from the m-Profiles and tested under compression following ASTM D695-10 (ASTM 2010). The average compressive modulus and strength are 28.89 GPa and 303.40 MPa, respectively.

![Figure 2 Photograph and cross-section of (a) m-Profiles and (b) slotted m-Profiles (dimensions are in mm)](image)

**Setup and Instrumentation**

The specimens were tested under monotonic axial compression using a 20,000 kN capacity universal testing machine. The loading was applied with a displacement control rate of 0.25 mm/min. As shown in Figure 3, the axial compressive test specimens were instrumented with eight linear variable differential transformers (LVDTs). Four of them (i.e. DT 1-4) are mounted on an aluminum cage attached to the mid-height of each specimen. The LVDT cage had a gauge length of 400 mm and it was placed at equal distance from each specimen end. Another four LVDTs (i.e. DT 5-6) were installed at the corners between the top and bottom steel clamps with a gauge length of 600 mm. The recorded deformations were used to correct the LVDT cage measurements. The aluminum cage unexpectedly broke due to the explosive failure of C-T4. But thankfully, the results from the experiments on specimen C-T4-4FPs, C-T4-4sFPs, and C-T4 show the measurements of the LVDTs installed between the top and bottom steel clamps are consistent with the LVDT cage measurements. Hence, for the remaining specimens, just the four LVDTs were installed at the corners between the top and bottom steel clamps were used. Additionally, four axial strain gauges (i.e. AG 1-4) having a gauge length of 10 mm were installed at the midheight of the FRP tube. Hoop strains around the specimen perimeter were measured by eight unidirectional strain gauges (i.e. HG 1-4) having a gauge length of 10 mm that were bonded horizontally on the FRP tube. For specimens with m-Profiles, a strain gauge was glued to each of m-Profile at midheight to measure its longitudinal strain, as shown in Figure 3.
Experimental results and Discussion

Experimental Observation and Failure Modes

All CFFT with m-Profiles specimens exhibited similar mechanical behavior. They exhibited a progressive failure process and have a relative good ductility. This is different from the sudden failure of CFFT specimens. The appearance of the typical specimens with four m-Profiles at different loading stages are showing in Figure 4. During testing, a few cracking noises were heard and followed a few white patches were observed on the FRP tubes. At this loading point the axial strains are around 0.012 for all specimens with four m-Profiles. Especially, as shown in Fig 4 (a), all those patches always appeared at the locations of FRP tube inside attached to m-Profiles. So this patches are potentially caused by the damage of m-Profiles. It will be further analyzed on the longitudinal strain of m-Profiles in the following section. As the test continued the local patches gradually widened and some new patches appeared. Then the load increased reaching their peak load levels. At peak load a catastrophic failure was not observed, just several white patches have developed to local fiber ruptured and FRP tubes still packaged the specimen but didn’t explosively ruptured, as shown in Figure 4 (b). After that specimens still had the carrying capacity. As the displacement continued to increase, accompanied with continuous snapping noise, the loads dropped as a result of a slow fracture of the FRP tube. The local fibers progressively ruptured and caused tearing failure of the FRP tube following its fiber orientation. Eventually, as the tearing developed throughout in the mid-height region and then some encased m-Profiles crushed (Fig 4 (c)), the specimens reached the ultimate states with a large drop in load occurring. Especially, it was very interesting to notice the crushed sections always located at the height corresponding where the first white patch appeared on the FRP tube. After ultimate state, the loads on Specimens with m-Profiles did not drop to zero unlike CFFT specimens. The axial loads in specimen with m-Profiles dropped to around 25% of the peak load, then tests were stopped.

As expected, the reference CFFT specimens C-T4 and C-T6 failed suddenly after reaching their peak load level, accompanied by an instantaneous loss of applied load. The loading processes of specimen C-T8 with the thickest FRP tube was terminated manually when the load reached to 12500 kN. Because the impact caused by the brittle failure at loads higher than 12500 kN beyond the loading capacity limitation of the testing facility.

Longitudinal Strain of m-Profiles

Figure 5 provides longitudinal strains of m-Profiles vs. the axial strains of the specimens from 0 to 0.016. It can be seen that the longitudinal stains in m-Profiles increased linearly with the axial stains of the specimens during the early stage of the loading with a slope closing to 45 degrees, which indicates the deformations of the m-Profiles and the specimen under axial compression are coordinate. Until the axial strain reached to 0.012~0.014, the longitudinal strain in m-Profiles no longer linearly increases with the axial strain of specimens with each drop in the curve representing the m-Profiles began to damage at this strain level. Now it is clear that the local white patches which appeared at this strain level were attributed to the damage in the m-Profiles. As the strain has already exceeded the ultimate strain from material tests, the material strength of m-Profile was fully used without bulking
occurring, which was benefit the inside support from concrete and outside confinement of FRP tube. The test was continued with an increase in load but a decrease in slope, representing the m-Profiles behaved a progressive failure after ultimate strain of itself.

Figure 4 Experimental observation of specimen C-T4-4FPs at initial failure (a), peak load, and ultimate state

Figure 5 Longitudinal strains of m-Profiles vs. the axial strains of the specimens

**Distribution of Hoop Strain**

Strain distributions shown in Figure 6 were established using hoop strains over the perimeter of the specimens recorded at a series of certain axial strain level up to the ultimate condition. It is shown the hoop strain distributions of specimens with m-Profiles all begin to exhibit irregular sharp after the strain beyond 0.014. As axial strain increased from 0.010 to 0.014, one of the strain values rapidly increased. For both specimens C-T4-4FPs and C-T4-4sFPs, one of the strains has already approached to 0.02 at axial strain of 0.014, following the specimens soon reached to their peak load just at strain of 0.0145 and 0.0157, respectively. For specimen C-T8-4sFPs having thicker FRP tube, the maximum strain at axial strain of 0.014 are around 0.016, also way higher than the other measured strains at the same time. Finally, due to the localized strain exceeded the rupture stain, FRP tube prematurely failed under the non-uniform distribution of hoop strains. By contrast, the distributions of CFFT specimens are very similar and all close to a circle shape under every certain axial strains, indicating the uniform distribution of lateral confining pressure. As expected, the FRP tube explosively ruptured due to this kind of uniform hoop straining.
Axial Load-Strain Behavior

The experimental results of the specimens are summarized in Table 1 with the axial load-strain curves shown in Figure 7 (a), where the axial strains of all specimens were based on the average axial shortening of the four LVDTs. It can be seen that CFFT with m-Profiles specimens exhibit similar axial load–strain behavior. The similar shapes of CFFT with m-Profiles specimens include a tri-linear ascending branches with a post-peak deformation segment. There are three critical point in each curve of specimens with m-Profiles, and all of them are marked as point (1), (2), and (3) in the Fig (a) and (b). The critical point (1) indicates the initial damage introduced by the damage occurring on internally m-Profile as the axial strain beyond its ultimate compressive strain. Before point (1), the curve of specimens with m-Profiles are similar to the bilinear ascending shape of CFFTs under compression. After that, the slope decreases due to the development of damage on internally encased m-Profiles but the load continues to increase and reach the peak point (2). From this point the damage developed to FRP tube and it started to rupture from the area where local stress concentration. The curve exhibited a post-peak deformation segment with a slowly decreasing load. The last critical point (3) indicates the ultimate failure, which appear as FRP tube tearing out, m-Profiles crushing and pushing out and concrete spalling. After that, the load rapidly drops to around 25% of the peak. The load and axial strain corresponding to each critical point for every specimen are listed in Table 2.

All the reference CFFT specimens have an approximately bilinear axial load-axial strain relationship in the loading process, as shown in Figure 7 (b), where the first portions of each are almost identical, but the slope of the second portion depends on the thickness of the thickness of FRP tubes. Both the curves of specimens C-T4 and C-T6 have a vertical descending segment, which is due to they failed suddenly after reaching its peak load. The loading processes of specimen C-T8 was terminated manually when the load reached to 12500 kN.
(a) CFFT with m-Profiles specimens        (b) Reference CFFT specimens

Figure 7 Axial load-strain relationships

### Table 2 Key Test Results

<table>
<thead>
<tr>
<th>Name</th>
<th>$P_{ini,f}$ (kN)</th>
<th>$\varepsilon_{ini,f}$</th>
<th>$P_{max}$ (kN)</th>
<th>$\varepsilon_{max}$</th>
<th>$\varepsilon_{rup}$</th>
<th>$P_{uf}$ (kN)</th>
<th>$\varepsilon_{uf}$</th>
<th>$P_{max,85%}$ (kN)</th>
<th>$\varepsilon_{max,85%}$</th>
<th>$DI$</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-T4-4FPs</td>
<td>7541.54</td>
<td>0.0112</td>
<td>7886.94</td>
<td>0.0159</td>
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<td>6703.99</td>
<td>0.0199</td>
<td>6703.99</td>
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<td>0.0124</td>
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<tr>
<td>C-T8-4FPs</td>
<td>9686.2</td>
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<td>10703.50</td>
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<td>C-T8-4sFPs</td>
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<td>C-T4</td>
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<tr>
<td>C-T6</td>
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<td>11755.00</td>
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<td>0.0205</td>
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<td>-</td>
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<tr>
<td>C-T8</td>
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<td>-</td>
<td>&gt;12513.5</td>
<td>&gt;0.0285</td>
<td>&gt;0.0162</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1*</td>
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</table>

**Influence of introducing m-Profiles**

It is evident to see from Figure 7 and Table 2 that the peak loads of specimens with m-Profiles are lower than that of their counterpart CFFT specimens without m-Profiles. The peak loads of specimens C-T4-4FPs, C-T4-4sFPs, are just 83~87% of specimen C-T4. And the peak loads of specimens C-T8-4FPs and C-T8-4sFPs are also below that of specimen C-T8. This is attributed to the damage on m-Profiles causing a stress concentration on the FRP tube. Consequently, FRP tube prematurely failed under non-uniform hoop straining. The test results shown in Table indicate that the rupture strain of specimens with m-Profiles at peak load is below 60% of their counterpart CFFT specimens. However, the specimens with m-Profiles have a relatively good failure ductile compared to the CFFT specimens. To evaluate the failure ductile of the specimens, the failure ductile index ($DI$) proposed by Han (2002) is used, which was calculated by:

$$DI = \frac{\varepsilon_{max,85\%}}{\varepsilon_{max}}$$

where $\varepsilon_{max,85\%}$ is the axial strain defined at an axial load corresponding to 85% of $P_{max}$ in the descending part of the load strain curve, and $\varepsilon_{max}$ is the axial strain corresponding to $P_{max}$. For those specimens without a descending part, $\varepsilon_{max,85\%}$ was the maximum strain. The obtained $DIs$ were listed in Table 2, which show all the $DIs$ of specimens with m-Profiles are bigger than 1, indicating the specimens have a better failure ductile than CFFT specimens. The slope of the second ascending branch is higher for the specimens with m-Profiles, which is due to the m-Profiles still maintain elastic in the initial stage of the second linear branch, meanwhile concrete has already reached its plastic state with decreasing of stiffness.

**CONCLUSIONS**

From the experimental test results and analysis presented in this paper, the following conclusions can be drawn:
Introducing m-Profiles in CFFT columns results in a decrease in the axial load-carrying capacity, which is due to the damage in the m-Profiles caused a stress concentration on the FRP tube and eventually caused its premature ruptured under non-uniform hoop straining. The test results indicate that the rupture strain of FRP tube at peak load is below 60% of their counterpart CFFT specimen without m-Profiles.

2 CFFT with m-Profiles under compression exhibit a progressive failure process with evident local white patches shown in FRP tube as warning before peak load, and has a good failure ductility with the progressive tearing failure of FRP tube after peak load. It is totally different from the catastrophic failure of CFFT specimens occurring without any evident warning before failure and any failure ductile after failure.

3 The material strength of internal m-Profiles is taken full advantage in compression, as the buckling is restricted by inside support from concrete and outside confinement from the FRP tube. The damage on m-Profiles encased develops until the axial strain beyond the ultimate strain of m-Profile itself.

ACKNOWLEDGMENTS

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ASTM. (2012). “Standard test method for apparent hoop tensile strength of plastic or reinforced plastic pipe by split disk method.” D2290-12, West Conshohocken, PA.
BEHAVIOUR OF HYBRID FRP/STEEL-CONCRETE BEAMS SUBJECTED TO STATIC FLEXURAL LOADING

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ABSTRACT

This paper provides the experimental results of a new hybrid beam intended for use in bridge applications. The hybrid beams were made up of hybrid Glass Fibre Reinforced Polymer (GFRP)/steel U-shaped section beams strengthened with a layer of concrete on top. Three hybrid FRP/steel-concrete beams were tested along with one control GFRP-concrete beam under four-point static flexural loading. It was concluded that the hybrid beams had higher flexural stiffness and strength than the control beam, where the beams reinforced with steel showed greater percent cost effectiveness than beams reinforced with CFRP. In addition, the improved connection mechanism used in the hybrid beams was found to provide adequate interface bond strength to maintain full composite action until ultimate failure.

KEYWORDS

FRP, steel-concrete, beam, flexural, hybrid

INTRODUCTION

Nowadays, extensive researches have been conducted for the application of fiber reinforced polymer (FRP) in civil engineering by utilizing its effective corrosion-resistance. Applications for strengthening and repair are already well established. A growing number of new bridges have been constructed as all-FRP or FRP-concrete hybrid structures [1-9], but also the shortcoming are obviously as higher initial costs and lower stiffness and poor ductility [10,11]. To overcome these drawbacks and to make the best use of materials, combinations of FRP, steel and concrete have been proposed. The advantages of hybrid structural systems include the improved stiffness and cost effectiveness based on material properties of each component.

In this paper, a new hybrid FRP/steel-concrete beam which is proposed. Three hybrid FRP/steel-concrete beams were tested along with one control GFRP-concrete beam under four-point static flexural loading to study the mechanical properties of the proposed hybrid beams.

HYBRID FRP/STEEL-CONCRETE BEAM DESCRIPTION

Although many configurations of hybrid FRP-concrete beams are possible, the proposed hybrid beam shown in Figure 1b, was chosen. The hybrid beam consists of a GFRP/steel U-shape hybrid beam, a GFRP cover plate and a layer of concrete on top. The U shape hybrid beam is compound with outer GFRP layers and inner U-shape steel section which could improve the stiffness of hybrid beam significant. CFRP layer which present more excellent properties than GFRP was not selected in this design due to its expensive cost and lacking of the source of carbon fiber in China at present. Our final objective is to develop a cost-competitive hybrid design comparing to traditional beam design, to suit the situation of China. The GFRP cover plate as formwork, where the concrete on, is anchored over the U-shape hybrid beam. To develop a good bond between GFRP beam and
concrete, 5~10 mm aggregate was applied to the cover plate with epoxy adhesive. Applying too much or too little aggregate could create insufficient bond between GFRP and concrete for the two materials to act compositely. The aggregate distribution percentage was recommended to be 35%~45% to obtain the optimal bond [12].

EXPERIMENTAL PROGRAM

The laboratory test presented in this paper should be considered as pilot test. The test was made to investigate the flexural behaviour of the proposed hybrid beam.

Test specimens

The experimental program consists of two 3500 mm long, one hybrid beam (Beam A) is the FRP U-shape without steel below, named GS0 and two hybrid beam (Beam B) are the FRP/steel U-shape below with the thickness of 2mm and 6mm steel plate, named GS2 and GS6. The dimensions of cross-sections of the two beams are shown in Figure 1.

Material properties

The main materials used in the work are GFRP, steel and normal concrete. The U-shape was made of inner Q235 steel and outer GFRP layers which fabricated by hand lay-up process at present trial stage. Two types of E-glass woven fabric were selected which were bidirectional E-glass woven fabric with a weight of 800 g/m² (0°/90°) and unidirectional fabric with a weight of 400 g/m² (0°), respectively. Vinyl-ester resin was chosen as the matrix. The cover plate had the architecture as [(0°/90°) 2, (0°), (0°/90°), (0°), (0°/90°), (0°)], while the GFRP U-shape and the GFRP/steel U had the architecture as [(0°/90°) 2, (0°), (0°/90°), (0°), (0°/90°), (0°), (0°/90°), (0°/90°)], and [(0°/90°) 2, (0°), (0°/90°), (0°), (0°/90°), (0°), (0°/90°), (0°), (0°/90°), (steel plate)]. The average fiber volume fraction of the GFRP part is about 0.3. The concrete layer on the day of test is 63.52 MPa. The specific material properties of the hybrid beam are recorded in Table 1.

<table>
<thead>
<tr>
<th>Type of material test</th>
<th>Module of elasticity (GPa)</th>
<th>Poisson ratio</th>
<th>Yield strength (MPa)</th>
<th>Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel plate</td>
<td>209</td>
<td>0.3</td>
<td>235</td>
<td>358</td>
</tr>
<tr>
<td>Longitudinal tension of cover plate</td>
<td>15.1</td>
<td>0.19</td>
<td>-</td>
<td>204.6</td>
</tr>
<tr>
<td>Longitudinal tension of U-shape</td>
<td>14.8</td>
<td>1.19</td>
<td>-</td>
<td>192.8</td>
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<tr>
<td>Concrete</td>
<td>32.4</td>
<td>0.21</td>
<td>-</td>
<td>63.52</td>
</tr>
</tbody>
</table>

Test set-up and apparatus

The test set-up is shown in Figure 2. Three beams were simply supported on rollers with a span length of 3300 mm and subjected to four point bending. The spacing between the inner loading points was 700 mm. The flexural test was performed by displacement control, and it was divided into two steps (shown in Figure 3). In the first step (Step I), a cyclic force profile was used with the amplitude gradually increased to examine the deflection stabilization. In the second step (Step II), force was increased monotonically until the test specimen failed.
EXPERIMENTAL RESULTS AND DISCUSSIONS

Load-deflection response and failure mode
The load-deflection curves at mid-span and the typical failure modes for all specimens shown in Figure 5 and Figure 6, respectively.

For Beam A, the load-deflection curve showed an almost linear-elastic response during Step I. In Step II, there was a slightly decrease in the slope of the curve due to the damage during Step I. The failure occurred at 241 kN when the concrete crushed inside one of the loading points as shown in Figure 6. After the failure of concrete, the continued loading caused the stress redistribution of the residual section which led to the web buckling of FRP U-shape finally.

For Beam B with two different thickness of steel plate, both the load-deflection curve also showed an almost linear-elastic response during Step I, which indicated a good behaviour of deflection stabilization. However, during Step II the slope of curve began to decrease gradually at this point. The failure occurred at 421.2 kN and 558.1 kN as the similar failure mode as Beam A, see Figure 6. The failure of the concrete was followed by the web buckling of GFRP/steel hybrid U-shape due to the poor capacity of webs in residual section after failing of concrete, shown in Figure 6.

Comparing to GS0, composting steel plate with the thickness of 2mm and 6mm has increased the flexural strength by 174.8% and 231.6%, while the stiffness by 250% and 460%. Thus, as the thickness of hybrid steel plate increase the bearing capacity and the stiffness could be improved significantly. It showed that the failure mode of hybrid GFRP-concrete beam and hybrid GFRP/steel-concrete beam were similar as concrete crushed first, followed the web buckling.

Longitudinal strain distributions
Figure 7 shows the mid-span longitudinal strain distributions through the depth of the three beams at different loads up to the ultimate load. The longitudinal strain distributions remained plane up to ultimate load and the strains increased closely linear with increasing load. The neutral axis was nearly the interface between the interfaces of concrete and U-shape, indicating the concrete was always in compressive state, which was design
to utilize concrete effectively. Meanwhile, as the increase thickness of steel plate the neutral axis were downward to the bottom. The development trends of mid-span longitudinal strain variations at the bottom and top surfaces of the three beams were similar to those of mid-span deflections. Figure 8 shows the yield strain through the depth of GS2 and GS6, it is notable that yield of steel developed from the bottom to the top as load increasing.

**Figure 7** Mid-span longitudinal strain distributions through the depth of Beams

**Figure 8** Yield depth of hybrid GFRP/steel concrete beams varies with load.

**CONCLUSIONS**

The following conclusions are deduced from the experimental results:
• As the increasing thickness of steel plate to hybrid GFRP-concrete beam both the stiffness and the flexural capacity of the section increasing significantly.
• Composite action between GFRP and concrete can be ensured using aggregate coating.
• The strain distribution across the depth of the hybrid beam remained linear until the beam fail. This confirmed the assumption that plane sections remains plane before and after bending.
• The failure mode of both hybrid GFRP-concrete beam and hybrid GFRP/steel-concrete beam were similar which was concrete crushed in compression followed by the web buckling.

**ACKNOWLEDGMENTS**

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Figure 3 Load history applied in the test

Figure 4 Locations of the displacement measurement

Figure 4 shows the positions of the displacement measurement to be acquired during the test. For all beams specimens, three linear displacements gauges shown in Figure 4, referred to as D1, D2 and D3 and electrical resistance strain gages were used to monitor the strain information of mid-span section of the three beams.

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Load-deflection response and failure mode

The load-deflection curves at mid-span and the typical failure modes for all specimens shown in Figure 5 and Figure 6, respectively.

For Beam A, the load-deflection curve showed an almost linear-elastic response during Step I. In Step II, there was a slightly decrease in the slope of the curve due to the damage during Step I. The failure occurred at 241 KN when the concrete crushed inside one of the loading points as shown in Figure 6. After the failure of concrete, the continued loading caused the stress redistribution of the residual section which led to the web buckling of FRP U-shape finally.

For Beam B with two different thickness of steel plate, both the load-deflection curve also showed an almost linear-elastic response during Step I, which indicated a good behaviour of deflection stabilization. However, during Step II the slop of curve began to decrease gradually at this point. The failure occurred at 421.2 kN and 558.1 kN as the similar failure mode as Beam A, see Figure 6. The failure of the concrete was followed by the web buckling of GFRP/steel hybrid U-shape due to the poor capacity of webs in residual section after failing of concrete, shown in Figure 6.

Comparing to GS0, composting steel plate with the thickness of 2mm and 6mm has increased the flexural strength by 174.8% and 231.6%, while the stiffness by 250% and 460%. Thus, as the thickness of hybrid steel plate increase the bearing capacity and the stiffness could be improved significantly. It showed that the failure mode of hybrid GFRP-concrete beam and hybrid GFRP/steel-concrete beam were similar as concrete crushed first, followed the web buckling.

Longitudinal strain distributions

Figure 7 shows the mid-span longitudinal strain distributions through the depth of the three beams at different loads up to the ultimate load. The longitudinal strain distributions remained plane up to ultimate load and the strains increased closely linear with increasing load. The neutral axis was nearly the interface between the interfaces of concrete and U-shaped, indicating the concrete was always in compressive state, which was design.
to utilize concrete effectively. Meanwhile, as the increase thickness of steel plate the neutral axis were downward to the bottom. The development trends of mid-span longitudinal strain variations at the bottom and top surfaces of the three beams were similar to those of mid-span deflections. Figure 8 shows the yield strain through the depth of GS2 and GS6, it is notable that yield of steel developed from the bottom to the top as load increasing.

Figure 8 Yield depth of hybrid GFRP/steel concrete beams varies with load.

CONCLUSIONS

The following conclusions are deduced from the experimental results:
• As the increasing thickness of steel plate to hybrid GFRP-concrete beam both the stiffness and the flexural capacity of the section increasing significantly.
• Composite action between GFRP and concrete can be ensured using aggregate coating.
• The strain distribution across the depth of the hybrid beam remained linear until the beam fail. This confirmed the assumption that plane sections remains plane before and after bending.
• The failure mode of both hybrid GFRP-concrete beam and hybrid GFRP/steel-concrete beam were similar which was concrete crushed in compression followed by the web buckling.

ACKNOWLEDGMENTS

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CONSTRUCTION OF A SHORT SPAN PEDESTRIAN BRIDGE USING HIGH CORROSION RESISTANT HYBRID FRP COMPOSITE BEAMS

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ABSTRACT
Corrosion is one of the main issues in the prestressed or steel bridges located in severe corrosive environments. Because of this, a large amount of money and time are required for maintenance and on renovation of bridges. Fiber reinforced polymer (FRP) is a good alternative to address this problem. Even though the HFRP is relatively expensive than the typical construction materials, it has advantages such as high corrosion resistance, low weight, and high tensile strength. This paper demonstrates the use of hybrid fiber reinforced polymer (HFRP) I-beams for construction of a short span pedestrian bridge in Japan. This bridge is the first HFRP short span pedestrian bridge in the world. It is located in a fishery harbor in Kure city, Hiroshima prefecture. The overall length and the width of the bridge are 12 m and 0.896 m, respectively. On-site loading tests were conducted on the bridge and it was satisfied the deflection criterion suggested by the Japan Society of Civil Engineers.

KEYWORDS
GFRP, Ultra-high strength fiber reinforced concrete, FRP bolts, Short span pedestrian bridge, Corrosion resistance, Bridge construction.

INTRODUCTION
During the fast economic growth period (after 1960) in Japan, a large number of short span pedestrian bridges were constructed. Most of those bridges were constructed using typical construction materials such as steel and prestressed concrete, so that the bridges located in marine environments can severely deteriorate by corrosion. Use of high corrosion resistant materials such as fiber reinforced polymers (FRP) for bridges can reduce the maintenance and renovation costs and time. In addition to the corrosion resistance, the FRP materials have high tensile strength, low weight, and high fatigue resistance. Even though the FRP bridges are associated with large initial cost, in terms of the life cycle cost, they are economical than the conventional construction materials (Nishizaki et al. 2006). And also, the use of the FRP for bridges is better than the steel or concrete as the FRP bridge construction relates with low emission of carbon dioxide (Tanaka et al. 2006).

This paper describes the construction of the first hybrid FRP (HFRP) short span pedestrian bridge in the world. This bridge was constructed in a fishery harbor in Kure city, Hiroshima prefecture, Japan in 2011. The main reason for construction of the HFRP bridge was to replace a deteriorated steel bridge and reduce the renovation and maintenance cost by the new FRP bridge. The steel bridge was used to access a pontoon from the pier and it was severely corroded and became unusable due to the marine environment. The new bridge consists of two HFRP I-beams topped with a glass FRP (GFRP) grating, which was used as the bridge deck. The overall length of the FRP bridge is 12 m and its width is 0.896 m. Figure 1(a) and (b) show the elevation and the cross section of the FRP bridge. The main girders of this bridge were made by joining two of 3 m long HFRP I-beams at both ends of a 6 m long HFRP I-beam. Different bolt types were used for the joints in order to select the best bolt type suitable for the marine environment. As all the structural components of the bridge can be pre-fabricated and transported to the construction site, these bridges have the advantage of rapid construction similar to steel bridges.
MATERIALS AND DESIGN PARAMETERS

Pultruded HFRP I-beam’s flanges consist of both the carbon fibers and the glass fibers and its web consists of glass fibers only. The main reason for having a GFRP web is to reduce the manufacturing cost of the I-beams. The overall width and the height of the HFRP I-beams are 95 mm and 250 mm and the thickness of flange and web are 14 mm and 9 mm, respectively. The details of the HFRP I-beam are shown in Figure 2. The mechanical properties of the HFRP flange and web are reported by the authors elsewhere (Wijayawardane et al. 2014). Hai et al. (2012) conducted four-point flexural tests on the HFRP I-beams and investigated the optimum carbon fiber content which will give the maximum flexural strength. According to that, the carbon fiber content for the HFRP I-beams was chosen for this study, which is 33% by volume.

![Diagram of HFRP pedestrian bridge](image)

Figure 1 Details of HFRP pedestrian bridge, Kure city, Hiroshima, Japan

In this pedestrian bridge, there were four types of beam-to-beam connections used to connect the HFRP I-beam parts together. The type of the beam-to-beam connection was differed by the number of bolts and the bolt type used for the connection. Used bolt types are 1) stainless steel bolts, 2) high corrosion resistant steel (HCRS) bolts, 3) stainless steel bolts with FRP cover, and 4) FRP bolts. Except the FRP bolts, all the other bolt types were made of steel with yield stress of 205 MPa and tensile strength of 520 MPa. The FRP bolts consist of glass fibers and they had tensile strength of 237 MPa, shear strength of 111 MPa, and tensile modulus of 23.7 GPa. The reason for using different bolt types is to understand the speed of deterioration of the bolts by the severe corrosive environment. Figure 3 shows the beam-to-beam connections used in the HFRP pedestrian bridge. The flexural behavior of each beam-to-beam connection having different bolt types was investigated by Hiroshi et al. (2016) and Hai et al. (2014). They conducted large-scale four point bending tests on the HFRP I-beams connected by above bolt types and selected the suitable bolt diameter and number of bolts required for each connection type. As shown in Figure 3, the I-beams were connected using 14 mm thick HFRP plates which are having a tensile modulus of 45.8 GPa.

In order to improve the bonding between the HFRP plates and the HFRP I-beams at the beam-to-beam connections, an epoxy adhesive (Sikadur-30) was used. The epoxy adhesive consist of two compounds and according to the manufacturer’s specification, it will have a shear strength of 10 MPa once it is cured under 20°C for 24 hours. The GFRP material was used for the bridge deck, stiffeners of the HFRP bridge girders, and handrails to enhance the corrosion resistance of the bridge and improve the durability.

The design guidelines provided by the Fisheries Agency in Japan (2003) was used to design the FRP pedestrian bridge. A design live load of 0.75 kN/m and a live load of 1.0 kN/m² was considered on the bridge girders and the bridge deck, respectively. Because of the low modulus of elasticity in FRP materials, the deflection limit becomes the dominant design criterion for FRP bridges. The Japan Society of Civil Engineers (2011) suggests that the
Deflection limitation for pedestrian bridges need to be $L/500$, where $L$ is the bridge span. According to that, the allowable deflection for the FRP pedestrian bridge in Kure city is 24 mm (12,000 m/ 500).

![Figure 2 Cross section of HFRP I-beam (units in mm)](image)

**CONSTRUCTION OF THE PEDESTRIAN BRIDGE**

The FRP bridge in Kure city was constructed very fast because its structural members were prefabricated in a factory and transported to the fishery harbor. In addition to that, there were low manpower and smaller space required for the construction of this bridge as the construction process was a matter of assembling the structural components together. During the fabrication of the HFRP I-beams and other FRP components, diamond bits were used to drill the bolt-holes to avoid excessive temperature increase in the resin matrix included in the FRP material. As shown in Figure 3, four bolt types were used at the beam-to-beam connections and the details of a joint is illustrated in Figure 5. A bolt-end distance of equal to or greater than $3d$ ($d = \text{nominal diameter of the bolt}$) was used, while the longitudinal and transverse spacing were equal to or greater than $4d$. The diameter of bolt-holes was 5% larger than the diameter of the bolts.

The construction sequence of the FRP short span pedestrian bridge is illustrated in Figure 4. At first, as shown in Figure 4(a), the HFRP I-beams were aligned on the supports. Then the epoxy adhesive was prepared by mixing its two compounds using an electric mixer (Figure 4(b)). An epoxy adhesive layer of approximately 0.5 mm to 1.0 mm was applied on the HFRP plates and on the HFRP I-beams at the joint location. It was made sure that
Figure 4 Timeline of assembly and installation of the FRP pedestrian bridge

(a) Align the HFRP I-beams on the support
(b) Mixing of epoxy adhesive
(c) Applying the epoxy adhesive on the HFRP plates and on the HFRP I-beam
(d) Filling the epoxy adhesive to the gap between consecutive HFRP I-beams
(e) Tightening the bolts of beam-to-beam connection
(f) Fixing stiffeners
(g) Fixing the GFRP grating
(h) Installing the bridge on the supports
(i) Completed FRP pedestrian bridge and the corroded steel bridge
the bolt-holes were not covered by the epoxy adhesive. The web-web joints were completed before the flange-flange joints. Figure 4(c) and Figure 4(d) shows the applying of the epoxy adhesive at a beam-to-beam joint. A higher torque of 20 N-m was applied to the stainless steel bolts, the HCRS bolts, and the stainless steel bolts with steel cover, whereas a lower torque of 10 N-m was applied to the FRP bolts (Figure 4(e)). This was due to the low tensile strength of the FRP bolts. The FRP bridge was fixed to the supports after finishing the fixing of GFRP stiffeners, installation of the GFRP grating, and fixing the handrails. The completed view of the FRP pedestrian bridge is shown in Figure 4(i).

Figure 5 Typical cross section of HFRP I-beam at joint (units in mm)

ON-SITE LOADING TEST

Before the installation of the FRP bridge, on-site loading tests were conducted to check the maximum deflection at midspan under the service load. As shown in Figure 6, two loading types were applied 1) concentrated load at midspan, 2) distributed load. The concentrated load was applied by ten number of people standing on the bridge and the loading details are given in Table 1. The load was increased by one person by one person getting on to the bridge and gathering at the midspan. In the case of the distributed load, the load was applied by eleven people and all the people were get on to the bridge at once (Figure 6(b)). The load applied under the distributed loading was 7.16 kN. When applying both loading types, mid-span deflection was measured using two LVDT transducers connected to the both girders of the bridge.

(a) Concentrated load at midspan  
(b) Distributed load along the centerline of the bridge

Figure 6 Loading types

<table>
<thead>
<tr>
<th>Table 1 Applied load on the bridge – Concentrated loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of people</td>
</tr>
<tr>
<td>------------------</td>
</tr>
<tr>
<td>Weight of people (kg)</td>
</tr>
<tr>
<td>Cumulative load (kN)</td>
</tr>
</tbody>
</table>

Figure 7 shows the relationship between the load and the deflection of each HFRP beam under the concentrated loading. Both beams showed almost same flexural behavior and the maximum deflection at midspan was approximately 22 mm. As recommended in the JSCE (2011) design guidelines, the maximum allowable deflection for this bridge is 24 mm. Therefore, the maximum deflection under the concentrated service load is less than the deflection limit (Figure 7). Under the distributed loading, both HFRP beams showed a deflection approximately
16 mm, which is significantly lower than the deflection limit. However, it is important to increase the stiffness of
the bridge in order to utilize the superior tensile capacity of the HFRP I-beams.

CONCLUSIONS

1. The HFRP I-beams can be used as the bridge girders in short span pedestrian bridges. Therefore, the durability
of the main girders will be increased and hence they can be used in severe corrosive environments.

2. Because all the components of this bridge are fabricated in a factory and transported to the construction site, the
construction time is significantly low as steel bridges. Furthermore, because of the ease of assembly of the bridge,
it will not require large manpower and high skilled labor.

3. The bridge was designed to meet the deflection limitation (L/500) suggested by the JSCE and it showed
satisfactory behavior under service loads. However, in order to utilize the high tensile capacity of the HFRP
material, the stiffness of the bridge need to be further improved.

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Tokyo, Japan, November.

Figure 7 Load-deflection relationship of the HFRP bridge girders under concentrated loading
Mini-symposium on FRP Sandwich Structures in Bridge and Building Construction

Organizer:
Thomas KELLER
VISCOELASTIC BEHAVIOUR OF COMPOSITE SANDWICH PANELS FOR CIVIL ENGINEERING APPLICATIONS: EXPERIMENTS AND ANALYTICAL MODELLING

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ABSTRACT

Composite sandwich panels comprising glass-fibre reinforced polymer (GFRP) faces, polyurethane (PUR) foam cores and longitudinal GFRP ribs are being increasingly considered for civil engineering structural applications. Similarly to other fibre reinforced polymer (FRP) components, composite sandwich panels are prone to creep and therefore this phenomenon must be duly considered in their design. However, experimental data about the viscoelasticity of this type of panels and their components is still scarce; furthermore, design guidelines for this particular phenomenon are still not available. This paper presents an experimental and analytical study about the creep response of GFRP-PUR composite panels. Flexural creep tests were conducted in full-scale panels, with and without longitudinal ribs, subjected to different load values. The ribs provided a considerable reduction in creep compliance compared to the simple panels. In order to analytically estimate the creep response of the full-scale panels, a composed creep modelling (CCM) approach is assessed. Therein, the creep behaviour of the individual constituent materials, determined from previous viscoelastic material characterisation studies, is used as input data for Timoshenko beam theory. The predicted and experimental creep curves presented good agreement for different load levels, attesting the adequacy of CCM for the analysis of creep in composite sandwich panels.

KEYWORDS

Composite sandwich panels, ribs, composed creep model, polyurethane foam, glass-fibre reinforced polymer.

INTRODUCTION

The use of composite sandwich panels for civil engineering structural applications is increasingly being considered by practitioners, as illustrated by the successful case studies regarding their use in bridge decks (Keller et al. 2014) and roof structures (Keller et al. 2008, Durand et al. 2012). Their potential for use in building floors has also been highlighted by several authors (Correia et al. 2012, Awad et al. 2012, Mousa and Uddin 2011), especially when the panels are reinforced with longitudinal ribs/webs (hybrid core) (Fam and Sharaf 2010, Correia et al. 2012). However, polymer-based composite sandwich panels, such as those comprising fibre reinforced polymers (FRP) and/or polymer foams, are prone to creep when subjected to significant permanent loads due to the viscoelastic nature of the polymeric materials (Huang and Gibson 1990, Garrido et al. 2014). This means that the effects of creep must be duly accounted for in the design of structural sandwich panels. To that end, data regarding the creep behaviour of sandwich panels and their constituent materials is necessary. Such data is still scarce and is often very much dependent on the sandwich panel architectures used in the corresponding experiments, meaning that their applicability in the analysis of different types of panels is limited.

This paper aims to address this issue by presenting an experimental assessment and analytical modelling of the creep response of two different types of composite sandwich panels, with and without reinforcement ribs. The uncoupling of the panels’ creep behaviour into the individual viscoelastic responses of their constituent materials is proposed. Such an approach, if successful in providing accurate estimates of creep deflections, allows the replacement of large scale creep testing on sandwich panels by small scale material characterisation tests. To this end, creep experiments are reported, carried out using sandwich panels comprising glass-fibre reinforced polymer (GFRP) faces, cores made of polyurethane (PUR) foam, and longitudinal GFRP reinforcement ribs/webs. A fairly simple and easily implementable composed creep modelling (CCM) approach is assessed, in which the creep behaviour of the individual constituent materials, determined from previous viscoelastic characterisation studies, is used as input data for Timoshenko beam theory to estimate the creep response of the full-scale panels. The
experimental creep curves obtained from those panels are compared with the model’s predictions for different load levels to evaluate the model’s accuracy.

LITERATURE REVIEW

Few studies have attempted to model the creep of full-scale sandwich panels using experimentally obtained viscoelastic properties of their constituent materials as a basis. Notable examples of such efforts may be found in the works of Huang and Gibson (1990), Shenoi et al. (1997), and Garrido et al. (2014). Huang and Gibson (1990) presented a study on the flexural creep behaviour of sandwich panels with aluminium faces and PUR foam cores with densities of 32, 48, 64, and 96 kg/m$^3$. The authors performed three-point bending creep experiments at various load levels up to 40% of the panels’ failure load for periods of 1200 h. The authors found that Timoshenko beam theory in combination with the time-dependent creep compliance of the foam cores provided reasonably good creep predictions, with deviations up to 10% from the experimental creep curves. Shenoi et al. (1997) studied the flexural creep response of sandwich panels comprising an FRP facing material and a rigid polyvinyl chloride (PVC) foam core. Sandwich beams were loaded in ten-point bending for periods up to approximately 1000 h. The obtained creep curves were used to estimate the creep compliance of the core material. Garrido et al. (2014) presented experimental and analytical investigations regarding the creep behaviour of a sandwich panel with GFRP faces and PUR foam core with density of 68 kg/m$^3$. The authors experimentally assessed the creep response of the PUR foam in shear, as well as the flexural creep response of the sandwich panel for a period of approximately 3600 h. Using a composed creep model, supported by a time-dependent shear modulus reduction function for the foam and an elastic modulus reduction function for the faces, the authors obtained creep deflection estimates based on Timoshenko beam theory. These compared well with the full-scale sandwich panel’s experimental creep curve.

To the authors’ best knowledge, there are no studies available in the literature regarding the creep of sandwich panels with reinforcement ribs/webs. These elements have shown high efficiency in improving the elastic behaviour (stiffness) of sandwich panels (Fam and Sharaf 2010, Correia et al. 2012). It would also be expectable that they would have a considerable influence in the creep behaviour of such panels.

EXPERIMENTAL PROGRAMME

Materials and Panel Architectures

In the current investigation, sandwich panels with GFRP face sheets were produced by vacuum infusion, moulded directly against the core material blocks. Two different panel typologies were adopted: (i) a simple cored panel with PUR foam, and (ii) a hybrid cored panel with PUR foam and two longitudinal GFRP ribs/webs along the panel edges. These were labelled as the “PUR” and “RIB” typologies, respectively. The mechanical properties of the materials used are given in Table 1. The cross-sectional dimensions of the sandwich panels are illustrated in Figure 1.

<table>
<thead>
<tr>
<th>Material</th>
<th>Layup</th>
<th>$E_L$ [GPa]</th>
<th>$E_T$ [GPa]</th>
<th>$G_{LT}$ [GPa]</th>
<th>$\nu_{LT}$ [-]</th>
<th>$\sigma_{\text{max},L}$ (T/C) [MPa]</th>
<th>$\sigma_{\text{max},T}$ (T/C) [MPa]</th>
<th>$\tau_{\text{max},LT}$ [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>GFRP</td>
<td>Faces $[0/0/30/-30/90/0]_S$</td>
<td>29.4</td>
<td>15.6</td>
<td>6.0</td>
<td>0.31</td>
<td>437 / 250</td>
<td>180 / 194</td>
<td>49.4</td>
</tr>
<tr>
<td></td>
<td>Ribs $[0/30/-30/0]_S$</td>
<td>22.2$^{(2)}$</td>
<td>15.2$^{(2)}$</td>
<td>2.3</td>
<td>0.30$^{(2)}$</td>
<td>-</td>
<td>-</td>
<td>34.8</td>
</tr>
<tr>
<td>PUR foam</td>
<td>Density [kg/m$^3$]</td>
<td>87.4</td>
<td>26.0</td>
<td>12.6</td>
<td>0.62 / 0.64</td>
<td>0.32</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(1) T: tension, C: compression; 
(2) properties estimated using Classical Laminate Theory (CLT); 
(3) through-thickness direction.

Figure 1 Cross-sectional dimensions of the sandwich panels: (a) PUR, and (b) RIB typologies
Experimental Setup

Flexural creep experiments were carried out for the two types of sandwich panel (Figure 2). The PUR panels were tested in a temperature controlled environment, with an average temperature of 19.9 °C ± 0.4 °C and relative humidity (RH) of 63% ± 5% (registered throughout the test duration). The RIB panels were tested in a different room with a high thermal inertia, for which an average temperature of 21.8 °C ± 0.7 °C and RH of 64% ± 6% was registered during the period of the creep experiments. The test durations were 790 h for the RIB panels and 910 h for the PUR panels. The panels were simply supported on steel rollers over a span of 3.30 m, and loaded using concrete slabs (each one with dimensions of 400 mm × 600 mm × 45 mm and weight of 0.26 kN) placed in six rows along the span to simulate a uniformly distributed load. Three specimens per panel type were tested, each with a different nominal load value \( p \): (i) 3.75 kN/m², (ii) 7.50 kN/m², and (iii) 15.00 kN/m².

![Figure 2 Full-scale creep tests setups for: (a) PUR panels, and (b) RIB panels](image)

EXPERIMENTAL RESULTS

The creep curves obtained per panel series are shown in Figure 3. Table 2 gives a detailed summary of the experimental results. This table also includes the exact values of the applied loads for each specimen, as well as a simple estimate of the shear stress percentage relative to the foam’s shear strength in the PUR panels. The elastic response of each panel series was consistent, with elastic compliance \( S_e \) values showing low scatter. The highest average values of this parameter were obtained for the PUR panels. A typical creep response, following a power law development with time, was obtained for the two panel types, as shown in Figure 3. Thus, individual power laws of the type given in Eq. 1 were fitted to the creep curves.

\[
\delta(t) = \delta_0 + m \left( \frac{t}{t_0} \right)^n
\]

In this equation, \( \delta \) is the total mid-span deflection, \( t \) is the time elapsed after load application, \( \delta_0 \) corresponds to the elastic mid-span deflection, \( m \) is the creep amplitude, \( n \) is the time exponent, and \( t_0 \) is the time unit considered. The creep amplitude and creep exponent values are given for each specimen in Table 2, as well as the obtained creep compliance values \( S_t = m/p \), where \( p \) is the applied load. The most pronounced creep response was observed in the PUR foam cored panels, which presented the highest average creep exponent and creep compliance values. The significantly lower creep response of the RIB panels compared to those with PUR foam confirms that
the addition of longitudinal GFRP ribs, besides significantly increasing the elastic stiffness, is quite effective in limiting the panels’ viscoelastic response.

The CCM approach provides predictions of a sandwich panel’s creep response by considering the creep of its individual materials/components and using them as input in an appropriate model of the panel’s structural behaviour. The effects of creep may be taken into account by considering the viscoelastic increase in the material’s deformations as time-dependent reductions ($\chi$) of its stiffness or elastic moduli. In the current study, this procedure was implemented using Timoshenko beam theory. Accordingly, Eq. 2 gives the mid-span deflection of a beam ($\delta^{L/2}$), considering the time ($t$) dependent effects of creep by: (i) affecting the Young’s modulus of the faces ($E_F$) by their respective reduction coefficient, $\chi_F$, and (ii) considering the sandwich panel’s shear stiffness, ($G_{AV}$)$_h$, as resulting from the contributions of the ribs and the foam core, given by ($G_{AV}$)$_h(t) = \chi_c(t)G_{cAV_c} + \chi_r(t)G_rA_{rAV_f}$. In this expression, $\chi_r$ and $\chi_c$ are the reduction factors for the ribs’ in-plane shear modulus ($G_r$) and for the shear modulus ($G_c$) of the core, respectively. In addition, $A_{rAV_f}$ and $A_{rAV_c}$ are the effective shear area of the ribs and core, respectively. Furthermore, in Eq. 2, $p$ represents the uniformly distributed load (load per unit length), $L$ corresponds to the span length, and $I$ is the second moment of inertia of the face sheets.

$$\delta^{L/2}_v(t) = \frac{5}{384} \frac{pl^4}{E_f(t)I} + \frac{1}{8} \frac{pl^2}{(G_{AV})_h(t)}$$

The moduli reduction coefficients for the faces, ribs, and core used in the current study are presented in the following section. Where temperature-dependent properties were available, the applicable test temperatures were taken into account in the modelling.

**Reduction Factors for Constituent Materials**

**PUR foam**

Garrido (2016) characterised the shear creep behaviour of a PUR foam similar to the one used in the current investigation. The author analysed and fitted the results using Findley’s power law (Findley 1960). In that study, the effects of temperature ($T$, in Kelvin) on that behaviour were also considered by extending the power law formulation to include creep parameters given by the Arrhenius equation. Based on the experimental characterisation of the foam’s viscoelastic response, shear modulus reduction factors were proposed for this foam, valid for temperatures between 20° C and 28° C, i.e., the range for which the model parameters were calibrated by the author. These reduction factors are given by the following equation, as a function of temperature and creep time:

$$\chi_c(t, T) = \left(1 + \frac{8.04}{246 \times 10^{-4} e^{\frac{12933}{8315}}}ight) \left(\frac{53.8 e^{\frac{12933}{8315}}}{15^{1.5}}\right)^{-1}$$
**GFRP faces**

In Garrido *et al.* (2016), a thorough characterisation of the flexural creep behaviour of GFRP laminates was carried out, using laminates similar to those of the face sheets in the sandwich panels of the current investigation. In this study the effects of temperature on the laminates’ creep response was also considered. The authors proposed the expression given in Eq. 4 to obtain the Young’s modulus reduction factors for that GFRP material, which is stress-dependent. However, this stress dependency was shown to only be significant for stress levels higher than ~25 MPa, a threshold that is well above the stress levels induced in the GFRP laminates in the current experiments.

\[
\chi_f(\sigma, t, T) = \left( 1 + \frac{27.6}{\sigma} \times 6.16 \times 10^{-13} e^{\left( \frac{-84498}{63167} \right) \sigma} \sinh \left( \frac{\sigma}{0.4} \right) t^{0.166} \right)^{-1}
\]  

**GFRP ribs**

Bottoni *et al.* (2014) carried out shear tests on GFRP specimens (produced by pultrusion, fibre layup not specified by the authors) at a controlled temperature of 20 °C and RH of 60%, for a period of about 760 days. The authors found that the shear creep deformations followed a typical power law development throughout the duration of the experiments. The time-dependent reduction factor given in Eq. 5 was determined from the creep data provided by the authors,

\[
\chi_r(t) = (1 + 0.149 \times t^{0.205})^{-1}
\]

**Model Predictions**

The predicted creep deflection curves are plotted in Figure 4 against the respective experimental results. One may observe that the creep predictions for the PUR panels reproduce quite well the overall development of the experimental curves. A slight overestimation of the creep deflections is obtained for the two lowest load levels. However, for the highest load, the CCM underestimates the experimental creep deflections, most likely due to the nonlinearity of the PUR foam’s creep behaviour at such high stresses. For this panel type, the CCM creep predictions are largely influenced by the creep response of the PUR foam, which exhibits significant viscoelasticity.

![Figure 4 Comparison between CCM predictions and experimental creep curves for the: (a) PUR panels, and (b) RIB panels](image)

In fact, considering the CCM predictions at 900 h of creep time, the creep deformations in shear account for 95% of the total creep deflection (for all load levels), while bending creep deformation accounts for only 5% of that deflection. Regarding the RIB panels, the CCM predictions slightly underestimate the experimental creep curves. However, the overall development of the predicted creep curves agrees well with the experimental results, attesting the adequacy of the CCM formulation in predicting the creep response of sandwich panels with longitudinal GFRP ribs.

**CONCLUSIONS**

Sandwich panels developed for civil engineering applications, presenting two different core solutions, namely (i) a simple PUR foam core, and (ii) a hybrid PUR foam and GFRP rib core, were experimentally and analytically studied regarding their flexural creep behaviour under uniformly distributed loading. The simple PUR foam cored panels exhibited high creep deformations, mostly due to the PUR foam’s marked viscoelasticity. The addition of longitudinal GFRP ribs to the panel core significantly reduced (three-fold) the creep compliance of the sandwich
panels, which presented considerably lower creep deformations for similar loads and time periods. The CCM predictions showed reasonable accuracy, further validating this approach for the prediction of the creep behaviour of full-scale sandwich panels, and extending its applicability to panels with longitudinal reinforcement ribs. The results obtained are particularly encouraging when taking into account the model’s relative simplicity and ease of use in predicting a very complex phenomenon such as the creep of sandwich panels.

ACKNOWLEDGMENTS

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REFERENCES


EXPERIMENTAL STUDY ON THE MECHANICAL BEHAVIOR OF FRP CURVED SANDWICH PANELS WITH FOAM CORE

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ABSTRACT

Non-linear architecture is becoming popular for its streamlined appearance and well fusion with environment. However, it faces construction difficulties because the linear formwork system is unsuitable for non-linear appearance. FRP (Fibre-Reinforced Polymer) curved sandwich panel with PU (Polyurethane) foam core, shows low density, high strength, convenient construction and excellent designability, which can satisfy the requirements for low cost and high speed in the construction of non-linear architecture. Additionally, thus sandwich panels can also act as the self-standing structures for its great mechanical properties, not only the formworks of the structures. However, there are little researches on the mechanical behavior and applications of FRP curved sandwich panels with PU foam core. This paper conducts a series of three-point flexural tests to study the mechanical behavior of such panels, including the influences of different geometrical or physical parameters on the load capacity and failure modes.

KEYWORDS

FRP, curved sandwich panel, flexural test, parameter study, local effect.

INTRODUCTION

The curved FRP sandwich panel with polyurethane (PU) foam core combines the advantages of FRP and PU foam, and provides high-strength, lightweight, anti-corrosion and easy-forming characteristics (Jones 1975). It is suitable for the non-linear and thermal-insulated architecture. However, there are little statically experimental studies on such curved panels (Bozhevolnaya and Kildegaard 1997, Frostig 1999, Keller et al. 2008), especially for the concentrated loading case, in which the local effects might be very obvious.

This paper conducts a series of three-point flexural tests to study the mechanical behavior of such panels, including the influences of different geometrical or physical parameters on the load capacity and failure modes.

EXPERIMENTAL PROGRAM

Material Properties

In this study, the glass fiber sheets were HITECH-G430S (Unidirectional fabric, 430 g/m²), which were produced by Nanjing Hitech Composites Co., Ltd. The resin system was SWANCOR 2511-1 A/BS epoxy resin and was produced by Swancor (Tianjin) Wind Blade Materials Co., Ltd. The PU foam was closed-cell and was produced by Kezhao New Materials Co., Ltd. The specimens were prepared using vacuum-assisted resin transfer molding (VARTM) process. The thickness of one single layer of FRP after solidification was approximately 0.45 mm, and the fiber mass fraction was approximately 68%. The material properties of GFRP (Glass Fibre-Reinforced Polymer) face sheet and PU foam core are shown in Table 1 and Table 2 according to Chinese standards GB/T 1447-2005 (FRP tension), GB/T 1448-2005 (FRP compression), GB/T 3355-2005 (FRP shear), GB/T 1452-2005 (PU tension), GB/T 1453-2005 (PU compression) and GB/T 1455-2005 (PU shear), respectively. The tension strength of GFRP face sheet in the 0° direction has not been obtained yet for the clamped failure of the coupon at the end of the tension process.
Table 1 Material properties of GFRP face sheet

<table>
<thead>
<tr>
<th></th>
<th>Tension</th>
<th></th>
<th></th>
<th>Compression</th>
<th></th>
<th></th>
<th>Shear</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$E$ (GPa)</td>
<td>$f_u$ (MPa)</td>
<td>$E$ (GPa)</td>
<td>$f_u$ (MPa)</td>
<td>$E$ (GPa)</td>
<td>$f_u$ (MPa)</td>
<td>$G$ (GPa)</td>
</tr>
<tr>
<td>$0^\circ$</td>
<td>38.9</td>
<td>not obtained</td>
<td>10.6</td>
<td>27.2</td>
<td>38.1</td>
<td>220.1</td>
<td>11.2</td>
</tr>
<tr>
<td>$90^\circ$</td>
<td>115.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 2 Material properties of PU foam core

<table>
<thead>
<tr>
<th>Density codes</th>
<th>Density (kg/m$^3$)</th>
<th>Tension $E$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>Compression $E$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>$G$ (MPa)</th>
<th>$f_u$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1</td>
<td>53.6</td>
<td>5.93</td>
<td>0.27</td>
<td>4.88</td>
<td>0.24</td>
<td>2.18</td>
<td>0.21</td>
</tr>
<tr>
<td>D2</td>
<td>77.6</td>
<td>12.52</td>
<td>0.58</td>
<td>11.34</td>
<td>0.50</td>
<td>4.96</td>
<td>0.3</td>
</tr>
<tr>
<td>D3</td>
<td>125.6</td>
<td>28.19</td>
<td>0.84</td>
<td>27.54</td>
<td>1.23</td>
<td>12.68</td>
<td>0.47</td>
</tr>
<tr>
<td>D4</td>
<td>168.5</td>
<td>43.16</td>
<td>1.30</td>
<td>47.96</td>
<td>2.23</td>
<td>20.81</td>
<td>0.65</td>
</tr>
<tr>
<td>D5</td>
<td>211.0</td>
<td>60.75</td>
<td>1.92</td>
<td>68.39</td>
<td>3.44</td>
<td>28.55</td>
<td>0.86</td>
</tr>
</tbody>
</table>

Table 3 Details of FRP sandwich panels with PU foam core

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Density (kg/m$^3$)</th>
<th>Inclined degree $\alpha$</th>
<th>Thickness of the foam $h_c$ (mm)</th>
<th>Stacking sequence</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1-30-20-A</td>
<td>53.6</td>
<td>30</td>
<td>20</td>
<td>0/90/0</td>
</tr>
<tr>
<td>D2-30-20-A</td>
<td>77.6</td>
<td>30</td>
<td>20</td>
<td>0/90/0</td>
</tr>
<tr>
<td>D3-30-20-A</td>
<td>125.6</td>
<td>30</td>
<td>20</td>
<td>0/90/0</td>
</tr>
<tr>
<td>D4-30-20-A</td>
<td>168.5</td>
<td>30</td>
<td>20</td>
<td>0/90/0</td>
</tr>
<tr>
<td>D5-30-20-A</td>
<td>211.0</td>
<td>30</td>
<td>20</td>
<td>0/90/0</td>
</tr>
<tr>
<td>D3-60-20-A</td>
<td>125.6</td>
<td>60</td>
<td>20</td>
<td>0/90/0</td>
</tr>
<tr>
<td>D3-30-15-A</td>
<td>125.6</td>
<td>30</td>
<td>15</td>
<td>0/90/0</td>
</tr>
<tr>
<td>D3-30-20-B</td>
<td>125.6</td>
<td>30</td>
<td>20</td>
<td>0/90/0/90/0</td>
</tr>
<tr>
<td>D3-30-20-C</td>
<td>125.6</td>
<td>30</td>
<td>20</td>
<td>0/90/0/90/0</td>
</tr>
</tbody>
</table>
| D3-30-20-D | 125.6            | 30                      | 20                               | 0/90/0/90/0      |}

Specimens

The specimens in this study include ten curved sandwich panels and one flat sandwich panel as Table 3 shows. In the specimen designations such as D1-30-20-A, the first part represents the density of PU foam core, as shown in Table 2, and the second part means the inclined degree $\alpha$ as shown in Figure 1, and the third part represents the thickness of the PU foam core, and the last part means the stacking sequence of FRP face sheet, in which the $0^\circ$ direction means that the fiber is parallel to the longitudinal direction of the specimen. The thickness of face sheet can be calculated from the thickness of a single layer. The details of each specimen are shown in Table 3.

Setup and Instrumentation

![Figure 1 Experimental setup and instruments for FRP sandwich panels with PU foam](image)

The typical test setups for FRP sandwich panels with PU foam core are shown in Figure 1. The loading was controlled by the displacement magnitude, and the loading rate was approximately 1 mm/min.
EXPERIMENTAL RESULTS

Experimental Observation and Failure Modes

The specimens D1-30-20-A, D3-60-20-A, D3-30-15-A, D3-30-20-C and D3-30-20-D showed similar mechanical behaviors. The PU foam core under the loading roller didn’t exhibit significant deformation during the loading process. When the maximum load was reached, shear failure suddenly appeared in the core and the load sharply decreased. The failure of this specimen was sudden and brittle, as shown in Figure 2(a).

The specimens D3-00-20-A, D3-30-20-A and D4-30-20-A exhibited similar mechanical behaviors and failure modes. The transversely compressive deformations of the PU foam core under the loading roller were not significant during the initial loading stages, but the indentations took place and rapidly developed when closing to the maximum load. When the specimens reached their maximum load, as Figure 2(b) showed, the face sheets under the loading roller yielded and the capacity decreased. For the specimen D3-00-20-A, the plastic hinge at the mid-span formed and the force transferring route changed from beam system into catenary system, and the lower face sheet started to take tension force. For the other two specimens, the core immediately showed shear failure after the face yielded and the capacity suddenly decreased.

The specimen D3-30-20-B showed no significant local deformation under the loading roller during the initial loading process. When the load reached 0.6\(P_{\text{max}}\) (the maximum load), the indentation formed and gradually developed. When the maximum load was reached, as Figure 2(c) showed, the upper face sheet close to the loading roller wrinkled and the capacity decreased a little bit. The core shear failure finally took place and the load sharply decreased when the loading process continued. The specimen D5-30-20-A also showed face wrinkling followed by core shear failure, but little indentation was observed during the load stages.

The specimen D2-30-20-A exhibited little indentation until the load reached 0.9\(P_{\text{max}}\). When the load reached approximately 0.9\(P_{\text{max}}\), as the Figure 2(d) showed, the indentation quickly developed and the load increased gradually. After large deformation of indentation, the core shear failure sharply took place.

Load-Displacement Relationships

The load-displacement curves are shown in Figure 3. The displacement is the value of the displacement transducer placed at the upper face sheet at the mid-span. The Figure 3(a) shows the comparison of relationships within different densities, indicating that the higher density provides higher bearing capacity and stiffness. The Figure 3(b) shows the comparison between the curved sandwich panels and the flat sandwich panel. It is obviously shown that the curved ones provide higher stiffness than the flat one. The Figure 3(c) shows the influences of different stacking sequences on the relationships, which indicates that more number of layers in 0° direction provides higher stiffness and changes the failure modes from indentation to pure core shear. The Figure 3(d) shows the comparison of different core thickness. It is found that the thinner core provides lower stiffness and leads the failure of pure core shear.

![Figure 2 Failure modes of the specimens](image-url)
The capacity of the specimen is effected by the density of the foam, the inclined degree, the stacking sequence and the core thickness. The details of bearing capacity are summarized in Table 4. It is found that the maximum load is controlled by the failure process. If the failure mode is core shear, the capacity can be predicted by considering shear failure of the foam core and the contribution of the GFRP face sheets. The failure modes are also summarized in Table 4. It is found that the indentation and the core shear are the two main failure modes when loaded in the mid-span with loading roller. The indentation can cause further failures such as face yield or face wrinkling. The core shear is usually the final failure mode if the loading process continues.

Table 4 Experimental results of the specimens

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Maximum load (kN)</th>
<th>Failure process</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1-30-20-A</td>
<td>0.507</td>
<td>Core shear</td>
</tr>
<tr>
<td>D2-30-20-A</td>
<td>1.351</td>
<td>Indentation, finally core shear</td>
</tr>
<tr>
<td>D3-30-20-A</td>
<td>2.195</td>
<td>Indentation, then face yield, finally core shear</td>
</tr>
<tr>
<td>D4-30-20-A</td>
<td>3.012</td>
<td>Indentation, then face yield, finally core shear</td>
</tr>
<tr>
<td>D5-30-20-A</td>
<td>3.968</td>
<td>Face wrinkling, finally core shear</td>
</tr>
<tr>
<td>D3-00-20-A</td>
<td>2.364</td>
<td>Indentation, then face yield, force transferring route changed</td>
</tr>
<tr>
<td>D3-30-20-A</td>
<td>2.176</td>
<td>Core shear</td>
</tr>
<tr>
<td>D3-30-15-A</td>
<td>1.689</td>
<td>Core shear</td>
</tr>
<tr>
<td>D3-30-20-B</td>
<td>1.295</td>
<td>Indentation, face wrinkling, finally core shear</td>
</tr>
<tr>
<td>D3-30-20-C</td>
<td>2.505</td>
<td>Core shear</td>
</tr>
<tr>
<td>D3-30-20-D</td>
<td>3.040</td>
<td>Core shear</td>
</tr>
</tbody>
</table>

The following conclusions can be drawn from this study:

- The capacity of the specimen is affected by the density of the foam, the inclined degree, the stacking sequence, and the core thickness.
- The maximum load is controlled by the failure process. If the failure mode is core shear, the capacity can be predicted by considering shear failure of the foam core and the contribution of the GFRP face sheets.
- The indentation and core shear are the two main failure modes when loaded in the mid-span with loading roller.
- The indentation can cause further failures such as face yield or face wrinkling. The core shear is usually the final failure mode if the loading process continues.
A series of three-point flexural tests are conducted to study the mechanical behavior of FRP curved sandwich panels with PU foam core. The influences of different geometrical or physical parameters on the load capacities and failure modes are concerned in the paper.

The curved sandwich panels show higher stiffness than the flat panel, and display different failure processes from the flat one. The force transferring route of the flat sandwich panel has changed for the forming of the plastic hinge in the upper face sheet at the mid-span.

The indentation and core shear are the two main failure modes in the tests. The local effects can’t be ignored when concentrated load exerting on such panels.

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THEORETICAL STUDY ON THE MECHANICAL BEHAVIOR OF FRP CURVED SANDWICH PANELS WITH FOAM CORE

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ABSTRACT

FRP (Fibre-Reinforced Polymer) curved sandwich panel with PU (Polyurethane) foam core combines the advantages of FRP and PU foam, showing low density, high strength, convenient construction and excellent designability. So it is a kind of structural components suitable for the need of low cost and fast construction when build non-linear architectures. However, there are little researches on the mechanical behavior of FRP curved sandwich panels with foam core. This paper develops an approach to analyze the mechanical behavior of such panels based on high order sandwich panel theory. The theoretical result is validated to be very precise in comparison with that of finite element analysis (FEA). With this method, the differences between the plain and the curved sandwich panel are compared and some parameter studies are further conducted.

KEYWORDS

FRP, curved sandwich panel, high order theory, finite element analysis, parameter study.

INTRODUCTION

With the development of non-linear architecture, the aims for easy construction are drawing more and more attention. Additionally, the continuing high energy consumption in buildings calls for better thermal insulation capabilities. Fibre-Reinforced polymer (FRP) is suitable to construct the non-linear architecture for its high-strength, lightweight, anti-corrosion and easy-forming characteristics (Jones 1975). The curved FRP sandwich panel with polyurethane (PU) foam core can not only fit the need of the non-linear architecture, but also significantly improve the thermal performance for the excellent thermal insulation capabilities of PU foam core. Various models have been developed to study the mechanical behavior of FRP sandwich panels (Noor et al. 1996, Carrera and Brischetto 2009). The classical and first-order model neglect the transverse deformation of the core (Carrera and Brischetto 2009). Frostig and co-workers have developed closed-form high-order sandwich panel theory (HSAPT) to analyze the flat and curved sandwich panel with compressible core without any preliminary deformational restrictions (Frostig et al. 1992, Frostig 1999).

This paper develops a simple but accurate enough method based on HSAPT to analyze the mechanical behavior of FRP cylindrical sandwich panels with foam core. The comparison of this method with finite element analysis (FEA) is conducted to assess the accuracy of this method. Some parameter studies are further conducted.

HIGH-ORDER SANDWICH PANEL THEORY FOR FRP SANDWICH PANELS

Mechanical Properties of FRP Curved Laminate

According to the classical laminate theory, the in-plane resultants of cylindrical FRP laminate are written as:

\[
\begin{bmatrix}
N_x \\
N_y \\
N_{xy}
\end{bmatrix} = \begin{bmatrix}
A_{11} & A_{12} & A_{16} \\
A_{21} & A_{22} & A_{26} \\
A_{61} & A_{62} & A_{66}
\end{bmatrix} \begin{bmatrix}
\epsilon_x \\
\epsilon_y \\
\gamma_{xy}
\end{bmatrix} + \begin{bmatrix}
B_{11} & B_{12} & B_{16} \\
B_{21} & B_{22} & B_{26} \\
B_{61} & B_{62} & B_{66}
\end{bmatrix} \begin{bmatrix}
\kappa_x \\
\kappa_y \\
\kappa_{xy}
\end{bmatrix}
\]

\[
\begin{bmatrix}
M_x \\
M_y \\
M_{xy}
\end{bmatrix} = \begin{bmatrix}
D_{11} & D_{12} & D_{16} \\
D_{21} & D_{22} & D_{26} \\
D_{61} & D_{62} & D_{66}
\end{bmatrix} \begin{bmatrix}
\epsilon_x \\
\epsilon_y \\
\gamma_{xy}
\end{bmatrix}
\]

(1)
where $A_{ij}, B_{ij}$ and $D_{ij}$ ($i, j=1,2,6$) are the stretching, coupling and bending stiffnesses, respectively. The strain and curvature matrices are written as follows:

\[
\begin{align*}
\hat{e}^e &= \begin{bmatrix} \hat{e}^e_r \\ \hat{e}^e_\varphi \\ \hat{e}^e_z \end{bmatrix} = \begin{bmatrix} u_r(\varphi,z) + w_z(\varphi,z) \\ v_\varphi(\varphi,z) \\ \frac{1}{r} \left( u_\varphi(\varphi,z) + \frac{1}{r} v_r(\varphi,z) \right) \end{bmatrix}, \\
\hat{e}^\varphi &= \begin{bmatrix} \hat{e}^\varphi_r \\ \hat{e}^\varphi_\varphi \\ \hat{e}^\varphi_z \end{bmatrix} = \begin{bmatrix} w_z(\varphi,z) - u_r(\varphi,z) \\ 2w_\varphi(\varphi,z) - u_\varphi(\varphi,z) \\ \frac{1}{r} \left( w_\varphi(\varphi,z) - \frac{1}{r} v_r(\varphi,z) \right) \end{bmatrix}.
\end{align*}
\]

(2)

where $u$, $v$ and $w$ are the displacements in the circumferential, the $Z$ and the radial directions, respectively.

**Mathematical Formulation**

The derivation of the mathematical formulation refers to the reference (Frostig 1999). The geometry and internal resultants and stresses are shown in Figure 1. The field equations are derived as follows.

For the FRP face sheets, the equations are:

\[
N_{r,\varphi} + \frac{M_{r,\varphi}}{r_i} - br_r(1 - \frac{d}{2r_i})\tau_{r,\varphi}(r = r_\text{in}, \varphi) = -n_r r_i + m_i
\]

(3)

\[
N_{b,\varphi} + \frac{M_{b,\varphi}}{r_b} + br_r(1 + \frac{d}{2r_b})\tau_{r,\varphi}(r = r_\text{out}, \varphi) = -n_r r_b + m_b
\]

(4)

\[
\frac{M_{r,\varphi}}{r_i} - N_r - br_r \sigma_{r,\varphi}(r = r_\text{in}, \varphi) + \frac{bd_r r_i}{2r_i} \tau_{r,\varphi}(r = r_\text{in}, \varphi) = -q_r r_i + m_{r,\varphi}
\]

(5)

\[
\frac{M_{b,\varphi}}{r_b} - N_b + br_r \sigma_{r,\varphi}(r = r_\text{out}, \varphi) + \frac{bd_r r_b}{2r_b} \tau_{r,\varphi}(r = r_\text{out}, \varphi) = -q_r r_b + m_{b,\varphi}
\]

(6)

where $N_j$ and $M_j$ ($j=t$ (i.e. top), $b$ (i.e. bottom)) are the in-plane resultants at the FRP face sheets, and $\sigma_{r,\varphi}(r=r_\text{in}, \varphi)$ and $\tau_{r,\varphi}(r=r_\text{in}, \varphi)$ ($j=t, b$) are the peeling and shear stresses in the interfaces, respectively.

For the foam core, the equations are:

\[
(r \tau_{r,\varphi}(r, \varphi))_r + \tau_{r,\varphi}(r, \varphi) = 0, \quad \tau_{r,\varphi}(r, \varphi) + (r \sigma_{r,\varphi}(r, \varphi))_r = 0
\]

(7)

Through the combination of kinematic relation, stresses fields, compatibility conditions and constitutive relations, the governing equation that only related to the core is derived as:

Figure 1 The geometry, the internal resultants and stresses of cylindrically curved FRP sandwich panel
FRP lamina and symmetric laminate

Because of the plane stress condition for HSAPT, the assumptions \(N_{y}^{f} = 0\) and \(N_{x}^{f} = 0\) are introduced, the load \(N_{f}^{o}\) is derived as Eq. 9 shows. The moment \(M_{f}^{o}\) can be similarly derived as follows.

\[
N_{f}^{o} = \left( A_{1} - \frac{2A_{1}A_{2}A_{6} - A_{2}^{2}A_{6} - A_{6}^{2}A_{12}}{A_{2}^{2} - A_{12}^{2}A_{6}} \right) \varepsilon_{o}^{0} \cdot M_{f}^{o} = \left( D_{11} - \frac{2D_{12}D_{26}D_{26} - D_{12}^{2}D_{66} - D_{26}^{2}D_{12}}{D_{11}^{2} - D_{26}^{2}D_{12}} \right) \kappa_{f}^{o} = (E)_{eq} \kappa_{f}
\]

Through the substitution of the Eq. 9 in the Eqs. 3-6, the four out of the five governing equations are derived as Eqs. 10-13 show. The fifth equation is listed as the above Eq. 8 shows.

\[
\left( \frac{(EA)_{eq}}{r_{i}} + \frac{(EI)_{eq}}{r_{i}^{3}} \right) u_{\phi,pp} + \frac{(EA)_{eq}}{r_{i}^{3}} w_{\phi,pp} - b r_{c} \left( 1 - \frac{d_{j}}{2r_{i}} \right) r_{i} = -n_{i} r_{i} + m_{i}
\]

\[
\left( \frac{(EA)_{eq}}{r_{b}} + \frac{(EI)_{eq}}{r_{b}^{3}} \right) u_{\phi,pp} + \frac{(EA)_{eq}}{r_{b}^{3}} w_{\phi,pp} + b r_{c} \left( 1 + \frac{d_{j}}{2r_{b}} \right) r_{b} = -n_{b} r_{b} + m_{b}
\]

\[
- \frac{(EA)_{eq}}{r_{c}} \frac{u_{\phi,pp}}{r_{c}} + \frac{(EI)_{eq}}{r_{c}^{2}} \frac{w_{\phi,pp}}{r_{c}^{2}} = \left( \frac{EA_{eq}}{r_{c}} \frac{w_{\phi,pp}}{r_{c}^{2}} - \frac{(EI)_{eq}}{r_{c}^{2}} \frac{w_{\phi,pp}}{r_{c}^{2}} \right) \varepsilon_{o}^{0} = -q_{c} r_{c} + m_{c,o}
\]

\[
- \frac{(EA)_{eq}}{r_{c}} \frac{u_{\phi,pp}}{r_{c}} + \frac{(EI)_{eq}}{r_{c}^{2}} \frac{w_{\phi,pp}}{r_{c}^{2}} = \left( \frac{EA_{eq}}{r_{c}} \frac{w_{\phi,pp}}{r_{c}^{2}} - \frac{(EI)_{eq}}{r_{c}^{2}} \frac{w_{\phi,pp}}{r_{c}^{2}} \right) \varepsilon_{o}^{0} = -q_{c} r_{c} + m_{c,o}
\]

\[
\kappa_{o,j}^{eq} = \kappa_{o,j}^{x} = 0
\]

Thus the in-plane resultants are written as:

\[
N_{f}^{o} = \left( A_{1} - \frac{A_{2}^{2}}{A_{12}} \right) \varepsilon_{o}^{0} + B_{1} \varepsilon_{o}^{0} - A_{12} \varepsilon_{o}^{0} = \left( A_{1} \varepsilon_{o}^{0} + B_{1} \varepsilon_{o}^{0} \right) \kappa_{f}^{eq} = \left( D_{11} - \frac{B_{12} B_{26}}{B_{16}} \right) \kappa_{f}^{eq} = (E)_{eq} \kappa_{f}
\]

The expressions for the governing equations are the same as the FRP lamina and symmetric laminates.

**Antisymmetric angle-ply laminates**

Since the variables \(u, w\) for HSAPT are only functions of \(\phi\), which means the coupling effects of FRP laminates are hard to be considered, the elements in Eq. 2 are in the forms:

\[
K_{o,j}^{eq} = 0, \quad \kappa_{o,j}^{eq} = 0
\]

**Asymmetric laminates**
The other cases except for the above types are usually denoted as asymmetric laminates. After some mathematical manipulations, the in-plane resultants are derived as follows with the plane stress assumption.

\[ N_j^\sigma = \left( A_1 - \frac{A_4 A_6 A_{26} - A_6^2 A_{26}}{A_{26}^2 - A_{26} A_{66}} A_2 A_6 A_{26} - A_6^2 A_{26} \right) c_{\sigma j}^\sigma + \left( B_1 - \frac{A_4 B_6 A_{26} - A_6^2 B_{26}}{A_{26}^2 - A_{26} A_{66}} A_2 B_6 A_{26} - A_6^2 B_{26} \right) \kappa_{\sigma j}^\sigma \]

\[ M_j^\sigma = \left( B_1 - \frac{A_4 B_6 B_{26} - B_6^2 B_{26}}{B_{26}^2 - B_{26} B_{66}} A_2 B_6 B_{26} - B_6^2 B_{26} \right) c_{\sigma j}^\sigma + \left( D_1 - \frac{A_4 D_6 B_{26} - B_6^2 D_{26}}{B_{26}^2 - B_{26} B_{66}} A_2 D_6 B_{26} - B_6^2 D_{26} \right) \kappa_{\sigma j}^\sigma \]  \hspace{1cm} (17)

Similarly, through the substitution of Eqs. 2 and 17 in Eqs. 10-13, the four out of the five governing equations can be derived. The fifth equation is still Eq. 8.

**FEA SIMULATION AND VALIDATION**

The specimens refer to the reference (Frostig 1999) as Figure 2 shows. The material properties are listed in Table 1 (Keller et al. 2010), where x, y and z correspond to the circumferential, the Z and the radial directions, respectively. The face sheets of symmetric laminate with the stacking sequence \([0/90/0]\), each layer of 0.5 mm, are first studied. The theoretical results are solved using software Maple and the finite element analysis (FEA) is conducted using ANSYS. The 20-node hexahedral solid element (solid 186) is used to simulate the PU foam and 8-node quadrilateral shell element (shell 99) is used to simulate the FRP face sheets.

![Figure 2 The geometry of the FRP curved sandwich panels with foam core](image)

<table>
<thead>
<tr>
<th>Materials</th>
<th>( E_x ) (GPa)</th>
<th>( E_y ) (GPa)</th>
<th>( E_z ) (GPa)</th>
<th>( G_{xy} ) (GPa)</th>
<th>( G_{yz} ) (GPa)</th>
<th>( G_{xz} ) (GPa)</th>
<th>( v_{xy} )</th>
<th>( v_{yz} )</th>
<th>( v_{xz} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>FRP lamina</td>
<td>31.8</td>
<td>3.8</td>
<td>3.8</td>
<td>3.5</td>
<td>3.5</td>
<td>3.5</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>PU foam</td>
<td>25e-3</td>
<td>10e-3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 3 shows that the HSAPT for FRP sandwich panels agrees well with the FEA. Figure 3(a) shows the radial displacements of the FRP face sheets. The largest error takes place at the mid-span, and is 1.61% for the upper face sheet (5.5000 mm with HSAPT and 5.4127 mm with FEA) and 1.11% for the lower face sheet (5.3206 mm with HSAPT and 5.3127 mm with FEA).
with HSAPT and 5.2624 mm with FEA). Figure 3(b) shows that the peeling stresses have stress concentration in the support vicinity. The simulation results of different stacking sequences of FRP face sheets are listed in Table 2. It is clearly shown that this method has enough accuracy.

<table>
<thead>
<tr>
<th>stacking sequences of each FRP face sheet</th>
<th>radial displacements of the upper face sheets</th>
<th>errors</th>
<th>radial displacements of the lower face sheets</th>
<th>errors</th>
</tr>
</thead>
<tbody>
<tr>
<td>[0°]</td>
<td>10.8525</td>
<td>10.5955</td>
<td>2.43%</td>
<td>10.6848</td>
</tr>
<tr>
<td>[0°/90°/0°/90°]</td>
<td>5.2268</td>
<td>5.1121</td>
<td>2.24%</td>
<td>5.0472</td>
</tr>
<tr>
<td>[30°/-30°/30°/-30°]</td>
<td>5.1868</td>
<td>5.1003</td>
<td>1.70%</td>
<td>5.0071</td>
</tr>
<tr>
<td>[45°/-30°/45°/-60°]</td>
<td>7.7274</td>
<td>7.5753</td>
<td>2.01%</td>
<td>7.5542</td>
</tr>
</tbody>
</table>

**PARAMETER STUDY**

The inclined degree is changed from 0 to 90, namely, from flat panel to semi-circle panel. The method developed in this paper is also suitable to the flat FRP sandwich panel with foam core. The results for uniform load case listed in Figure 4. It is clearly seen that with the increase of the inclined degree, the mechanical parameters under uniform load all decrease quickly before about 30 degree, and decrease gradually after that until 90 degree. In comparison with the curved sandwich panels, the flat panel has the largest deflection, peeling stresses and shear stresses in the interfaces under the same load and boundary conditions. Figures 4(b) indicates that the local effects are located in the vicinity of the support, and decrease when the inclined degree increases.

**CONCLUSIONS**

The main conclusions are as follows:

(1) Based on HSAPT, an analytical method is put forward to analyze mechanical behaviors of the FRP cylindrically curved sandwich panels with foam core.

(2) This method is validated to be accurate enough compared to the results of the FEA. The method is suitable for different stacking sequences for the FRP face sheets, but the coupling effects are not calculated.

(3) The flat sandwich panel has the much larger deformation, peeling stresses and shear stresses compared to the curved panels under the same load and boundary conditions.

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**REFERENCES**


THE EFFECT OF FOAM CORE DENSITY ON THE FLEXURAL BEHAVIOUR OF SANDWICH PANELS WITH FLAX FRP SKINS

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ABSTRACT

The use of composite sandwich panels as structural elements is becoming increasingly widespread due to their high strength-to-weight ratios and favourability for pre-fabricated construction methods. This study analyses the effect of foam core density on the flexural behaviour of sandwich panels with novel bio-composite unidirectional flax fibre-reinforced polymer skins. Nine 1000x100 mm² sandwich panels with symmetrical skins and a 50 mm thick core were fabricated and tested in four-point loading. This experimental program compares the effect of three polyisocyanurate foam cores with varying densities when used in conjunction with three-layers of flax-FRP for each skin.

KEYWORDS

Bio-based, sandwich panel, flax fibres, foam core, density, skin.

INTRODUCTION

The recent movement towards sustainable and environmentally friendly building practices has brought the development of alternative building materials and methods to the forefront of current research. As a result, the popularity of highly efficient buildings and pre-fabricated modular construction has risen considerably to meet new sustainability design criteria. Sandwich panels are an example of highly efficient systems that can be optimized for rapid-construction for the cladding of buildings or lightweight decking.

Sandwich panels are composite systems comprised of two stiff skins on opposite faces of a core material. The core material is often comprised of lightweight foam polymer, balsa wood, or aluminium honeycomb structure. Structural Insulated Panels (SIPs), a denomination of sandwich panels, are typically selected based on insulating properties. The most common foam cores are closed-formed cell structures such that they are impermeable to water and are isotropic, including polyurethane (PUR), polyvinylchloride (PVC) and polyisocyanurate (PIR) (Carlsson, 2011). As a result, sandwich panel systems boast a high strength-to-weight ratio.

Although typical skin materials include concrete, sheet metal and oriented strand board, fibre-reinforced polymers (FRP) have been implemented as the skin materials in SIPs and have shown promise in providing high structural capacity. Glass fibre-reinforced polymer (GFRP) skins are the most common type of FRP skin; however, they rely heavily on synthetic materials. Mak et al (2015) demonstrated the promise of natural flax fibres as a more sustainable alternative to commercially established GFRP, demonstrating that three layers of unidirectional flax fibres are equivalent in strength and stiffness to a single layer of GFRP.

Mathieson and Fam (2008) investigated the static behaviour of sandwich panels with GFRP skins and soft PUR foam cores and found that the core material governs in design due to significant shear deformations comprising up to 93% of the total deflection. Sharaf et al. (2010) investigated the effects of core density on one way bending of sandwich panels using two PUR foam cores and GFRP skins. It was found that increasing the density of the core material significantly increased the stiffness and strength of the composite system, while reducing the degree of core shear deformations. The governing failure mode of thin-skinned sandwich panels is often due to inward wrinkling of the skins, shear failure of the foam core or debonding at the core-skin interfaces. Sharaf et al. (2010) found lower density foam cores suffer from localized inward wrinkling of the compression GFRP skin. Mak et al.
(2015) showed that this failure type was consistent in sandwich panels comprised of three layers of unidirectional flax fibre-reinforced polymer (FFRP) skins, and that the ultimate capacity of three-layered FFRP skinned sandwich panels matches closely to that of identical GFRP sandwich panels.

PIR foam cores were used in this study due to low thermal conductivity for cladding applications. Applications involve spray foam and rigid foam boards. Spray foam cores were investigated by Shawkat et al. (2008); however, panels demonstrated poor quality control. Mak et al. (2015) studied the performance of sandwich panels with rigid foam boards using different manufacturing methods. A reduced ultimate strength was reported with the use of vacuum bag moulding as opposed to wet layup moulding using rigid foam.

This study investigates the effect of PIR foam core density on the flexural behaviour of FFRP sandwich panels subjected to four-point bending. Sandwich panels with skins comprised of three-layered unidirectional FFRP were constructed using three separate PIR foam cores of increasing density. The panels were constructed using a commercially available epoxy. The impact on strength, stiffness, and failure mode with changes in core density are examined.

MATERIAL SPECIFICATIONS AND TESTING

Materials

This section highlights the properties for all materials used in the fabrication of the test specimens.

Flax Fibres: Unidirectional flax fibre fabric was used with a reported tensile strength and modulus of 500 MPa and 50 GPa respectively, and an ultimate elongation of 1.5%. The dry density of the fibres is 1.28 g/cm³ (Composites Evolution Ltd. 2012). Mak et al. (2015) found the tensile and compressive strengths of four layers of unidirectional flax fibres with Tyfo S epoxy to be 150 MPa and 77 MPa respectively, and the tensile and compressive moduli to be 8.7 MPa and 4.2 MPa respectively.

Epoxy: Tyfo S, a two-component high elongation epoxy resin was used as the saturating resin. The reported tensile strength and modulus are 72.4 MPa and 3.18 GPa respectively, the compressive strength and modulus are 86.2 MPa and 3.2 GPa respectively, and the maximum elongation is 5%. These reported material properties are given after a 72 hours post cure at 60°C (Fyfe Co. LLC., 2012).

Foam: Three separate rigid, unfaced, closed cell polyisocyanurate foams were used: ELFOAM P200, P400, and P600 with respective densities of 32, 64, and 96 kg/m³. The P200 foam has a reported parallel and perpendicular shear strength of 151 kPa and 110 kPa, parallel and perpendicular shear modulus of 1.52 and 1.22 MPa, and an R-value of 1.06 m²°C/W (Elliott Company, 2012). The reported density is 32 kg/m³. The P400 foam has a reported parallel and perpendicular shear strength of 379 kPa and 344 kPa, parallel and perpendicular shear modulus of 5.86 and 5.17 MPa, and an R-value of 1.04 m²°C/W (Elliott Company, 2012). The reported density is 64 kg/m³. The P600 foam has a reported parallel and perpendicular shear strength of 585 kPa and 489 kPa, parallel and perpendicular shear modulus of 7.23 and 6.06 MPa, and an R-value of 0.97 m²°C/W (Elliott Company, 2012). These directions are representative of the direction of manufacturing. The reported density is 96 kg/m³. The density of the foam used in this study was calculated to be 92.7 kg/m³.

PIR Foam Core Tests

Compression tests

Five coupons were tested for each foam density according to ASTM C365 (ASTM 2011). Each coupon was cut into a 50-by-50-by-50 mm³ cube. The coupons were placed between two steel compression plates in an Instron 8802 testing machine. The compression tests were conducted at a rate of 1.0 mm/min until 80% strain was reached. A linear potentiometer (LP) was used to measure the total displacement between the compression plates and related to the strain in the sample. For each density, the initial response of the PIR foam was linear until reaching a plastic plateau after initial crushing, followed by strain hardening. The response of each density can be seen in Table 1 and Figure 1.

Tension tests

Five coupons were tested for each foam density according to ASTM C297 (ASTM 2016). Each coupon was cut into a 50-by-50-by-50 mm³ cube. The coupons were bonded to steel T-sections with epoxy resin to be gripped using an Instron 8802 testing machine. The tension tests were conducted at a rate of 0.5 mm/min until failure. For
each density, the response of the PIR foam was linear until failure. Strain in the sample was related to the displacement between the steel grips. The response of each density can be seen in Table 1 and Figure 1.

![Stress-strain graph](image)

Figure 1 Representative stress-strain response for the 32, 64 and 96 kg/m³ densities of PIR foam core in compression and tension, where compressive strains are shown here as negative, and tension positive.

<table>
<thead>
<tr>
<th>Foam Type</th>
<th>Calculated Density (kg/m³)</th>
<th>Compression Yield Stress (MPa)</th>
<th>Compression Elastic Modulus (MPa)</th>
<th>Ultimate Tensile Strength (MPa)</th>
<th>Tensile Elastic Modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>32</td>
<td>31.2</td>
<td>0.209 ± 0.008</td>
<td>4.89 ± 0.27</td>
<td>0.198 ± 0.029</td>
<td>10.1 ± 1.5</td>
</tr>
<tr>
<td>64</td>
<td>62.4</td>
<td>0.456 ± 0.002</td>
<td>12.60 ± 0.11</td>
<td>0.317 ± 0.079</td>
<td>20.0 ± 0.4</td>
</tr>
<tr>
<td>96</td>
<td>92.7</td>
<td>0.869 ± 0.005</td>
<td>35.1 ± 1.32</td>
<td>0.568 ± 0.197</td>
<td>59.3 ± 3.2</td>
</tr>
</tbody>
</table>

Fabrication of Sandwich Panels

Nine sandwich panels were fabricated using the wet lay-up method for manufacturing composite materials. Rigid sheets of 1050x350 mm² of each foam density were set on plastic sheeting and dabbed with a damp towel to remove any particles that may impact the core-skin bond. The foam was evenly wetted with epoxy resin. A sheet of flax fibre was placed along the foam sheet and additional epoxy resin was added until the installed fibres were saturated. The processes was repeated until three saturated layers were in place. A sheet of plastic was placed over the completed side and the panel was flipped over to apply three layers to the opposite side. Finally, the excess epoxy resin and air was worked out of the skin. A large plastic panel was placed on the sandwich panel. Steel plates were then placed on top of the large plastic panel during the curing process to ensure smooth skins of even thickness. Each completed sandwich panel was then cut using a circular saw into three 1000x100 mm² repetitions for testing, denoted by A, B and C. Each type of specimen was given a unique title to reflect the comprising materials. For instance, F64 refers to the three repetitions of flax skinned (F) panels with a 64 kg/m³ foam core. All specimens were allowed to cure at room temperature for a minimum of a month prior to cutting and testing.

Test Setup and Instrumentation

The specimens were tested in a Lab Integration universal testing machine in simply supported four-point loading at a rate of 2 mm/min, as shown in Figure 2. A 3 mm thick rubber pad was placed between steel plates at the end supports and loading points in order accommodate small rotations and minor various in skin thickness. The total span of each specimen was 900 mm, including two 375 mm long shear spans outside of the 150 mm constant moment region at mid-span. One electrical resistance strain gauge was adhered to each skin at mid-span in order to measure longitudinal skin strains. Three linear potentiometers (LPs) were placed at mid-span against aluminium extenders attached to opposite sides of the tension skin in order to measure deflections and rotations.
EXPERIMENTAL RESULTS AND DISCUSSION

Flexural Behaviour

The load-deflection plots for each sample repetition for each foam core density are seen in Figure 3. The load-deflection curves have two components: an initial linear response, followed by a non-linear, more ductile response until failure. No significant rotations were seen at mid-span and were therefore considered negligible. Figure 3 shows the load-strain response in both the compression and tension skins in the longitudinal direction, where compressive strains are negative. A summary of the test results is shown in Table 2.

Figure 3 Load-deflection (top row) and stress-strain (bottom row) for each sandwich panel repetition (A/B/C) with three-layer unidirectional FFRP skins and 32 (left), 64 (centre) and 96 kg/m³ (right) PIR foam core, where compressive strains are presented as negative and tension positive.
Table 2 Test Matrix and Results for Flexural FFRP Sandwich Panels, where the failure modes are presented as W for inward skin wrinkling in a shear span, S+D for primary core shear failure in a shear span followed by secondary debonding of both tension and compression skins, and C for skin crushing in a shear span

<table>
<thead>
<tr>
<th>ID</th>
<th>Foam density (kg/m³)</th>
<th>Repetitions</th>
<th>Ultimate load (kN)</th>
<th>Standard deviation (kN)</th>
<th>Stiffness (kN/m)</th>
<th>Standard deviation (kN/m)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>F32</td>
<td>31.4</td>
<td>3</td>
<td>1.41</td>
<td>0.04</td>
<td>62.17</td>
<td>0.95</td>
<td>W, W, W</td>
</tr>
<tr>
<td>F64</td>
<td>62.4</td>
<td>3</td>
<td>2.54</td>
<td>0.06</td>
<td>102.32</td>
<td>1.17</td>
<td>W, W, W</td>
</tr>
<tr>
<td>F96</td>
<td>92.7</td>
<td>3</td>
<td>4.34</td>
<td>0.37</td>
<td>155.38</td>
<td>8.65</td>
<td>S+D, S+D, C</td>
</tr>
</tbody>
</table>

Failure Modes

The primary failure mode for all repetitions of 32 and 64 kg/m³ density foam core samples was inward wrinkling of the compression skin in the shear span near the loading points. This failure mode is a result of localized compression failure of the foam while bracing the skin. The failure modes observed can be seen in Figure 4. The compressive yield stresses of each foam density were compared to the ultimate stress applied to the panels at each loading point. This assumes ideal load transfer. The 32, 64 and 96 kg/m³ PIR failed at 87%, 72%, and 65% of their respective compressive yield stresses. This further explains the shift in failure mode away from inward skin wrinkling in samples with high density cores. Failure before the ultimate compressive capacity of the core is attributed to additional stresses due to bending.

The increase of density to 96 kg/m³ demonstrated a shift in the governing failure mode to core shear failure in the shear span. This resulted in an increased shear transfer between the core and skin, as well as more effective bracing of the skins. As a result, the shear capacity of the core material became the limiting factor in terms of ultimate strength for two of the three repetitions, despite one of those repetitions reaching a higher ultimate load. The third repetition showed crushing of the compression skin in the shear span, followed by debonding of the shear span skin. The compressive strains of the FRP at failure matches closely to the ultimate compressive strain of 0.0153 for four-layered FFRP skins (Mak and Fam 2015), therefore failure due to skin compression is expected.

![Figure 4](image1.png)

Three failure modes are inward skin wrinkling in the shear span (left), skin crushing in the shear span (centre), and primary core shear failure in the shear span followed by secondary debonding of the skin (right)

![Figure 5](image2.png)

The effect of PIR foam core density on the ultimate strength (left) and initial stiffness (right) of sandwich panels with three layers of unidirectional FFRP skins.
Effect of Foam Core Density

Figure 5 shows the average increase in strength and stiffness observed by increasing the density of the foam core. Taking the lowest core density as a baseline, doubling the foam core density from 32 kg/m$^3$ to 64 kg/m$^3$ resulted in increases of 82% and 65% for ultimate strength and initial stiffness respectively. Tripling the core density from 32 kg/m$^3$ to 96 kg/m$^3$ resulted in increases of 213% and 150% for ultimate strength and initial stiffness respectively. The stiffness of each sample was taken as that seen in the initial linear response.

CONCLUSION

This study examined the effect of PIR foam core density in sandwich panels with FRP skins comprised of three layers of unidirectional flax fibres in four-point bending. The following conclusions were drawn:

1. Increasing the density of the PIR foam core resulted in significant increases in ultimate capacity. Doubling the density from 32 kg/m$^3$ to 64 kg/m$^3$ and tripling the density from 32 kg/m$^3$ to 96 kg/m$^3$ saw increases in strength of 82% and 213%, respectively.

2. Increasing the density of the PIR foam core resulted in significant increases in stiffness. Doubling the density from 32 kg/m$^3$ to 64 kg/m$^3$ and tripling the density from 32 kg/m$^3$ to 96 kg/m$^3$ saw increases in stiffness of 65% and 150%, respectively.

3. The 32 and 64 kg/m$^3$ densities saw inward skin wrinkling in one of the two shear spans as the primary failure mode. For the 96 kg/m$^3$ density core, this failure mode transitioned to primarily core shear failure, followed by secondary debonding of the skin in one of the two shear spans.

4. The 96 kg/m$^3$ core’s shear modulus provided adequate skin bracing and shear transfer to allow the FFRP skins to approach their expected ultimate compressive capacity.

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REFERENCES


FRP SANDWICH STRUCTURES: CASE STUDIES IN AUSTRALIA

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ABSTRACT

The application of fibre reinforced polymer (FRP) composites in civil infrastructure has substantially increased around the world in the last two decades. Breakthrough research and developments carried out in Australia on FRP sandwich panels over the years have paved way to diverse applications including new structures and structural rehabilitation. This paper highlights some of the innovative applications of FRP sandwich structures in civil infrastructure in Australia. Selected case studies include a) composite bridge decks and girders, b) transoms in railway bridges, c) replacement sleepers in railway turnouts and d) innovative sleepers in mainlines. The challenges faced in applying this emerging technology in civil construction will be discussed including the need for appropriate education and training of engineers. An overview on the future potential of FRP sandwich structures in the construction industry will be also discussed.

KEYWORDS

Sandwich panel, bridge decks, railway sleepers, composite structures, technological developments, infrastructure applications.

INTRODUCTION

Fibre composite sandwich systems as primary load-bearing elements have been increasingly used in civil infrastructure in the last decade. A structural sandwich is a special form of a laminated composite fabricated by attaching two thin but stiff skins to the lightweight but thick core (ASTM C274-99). Because of this special feature, the sectional area is increased and consequently an increase in its flexural rigidity. The strength of this type of construction results from the combination of properties from the skin, core and interface. In a sandwich structure, the strong and stiff skins carry most of the in-plane and bending loads while the core mainly bears the transverse shear and normal loads (He and Hu 2008). Fibre composites are now commonly used for the top and bottom skins due to its high mechanical performance and low density. On the other hand, the core provides a sandwich construction with high flexural stiffness and strength with a relatively lightweight structure (Galletti et al. 2008). The interaction of the inherent properties of these constituent materials makes composite sandwich construction an efficient structural system.

Over the last few years, the development in fibre reinforced polymer (FRP) composite sandwich structures has been very exciting in volumes as well as in applications. The evolution of sandwich structures with enhanced material systems (skin and core) provided an opportunity to expand the application of this material in civil infrastructure. At present, there is a strong interest in the development and applications of fibre composite sandwich structures for civil and building material systems. Some examples of structural applications of sandwich composites include the first balsa cored composite bridge deck installed in Louisiana (SAMPE 2010), web for an innovative hybrid box beam section (Canning et al. 1999) and the function integrated GFRP sandwich roof structure (Keller et al. 2007). In these applications, sandwich composites were adopted because of their lightweight which facilitates handling during assembly, and reduces installation and transportation costs. They also offer corrosion resistant structures requiring less maintenance. In Australia, fibre composite sandwich structures are now being used as structural panels, building components in residential and industrial buildings, and as bridge decks in a number of infrastructure projects. This paper presents an overview of the recent developments and initiatives on fibre composite sandwich structures, which have been evolving and have become a viable construction material in civil engineering and construction. The on-going research, development and applications of a novel composite sandwich panel made up of glass fibre reinforced polymer skins and a modified phenolic
core material at the Centre for Future Materials (CFM) at the University of Southern Queensland (USQ) are also highlighted.

THE NOVEL SANDWICH PANEL

A novel sandwich panel comprises a proprietary phenolic core material sandwiched between two high strength glass fibre reinforced skins as shown in Figure 1. In the production of the panel, the dry fibres for the top and bottom skins are infused with resin at the same time and co-cured with the phenolic core. The solid phenolic core has been purposely designed to have high compressive and shear strength to carry the concentrated loads typical for bridges and other structural elements. The sandwich panel is strong, light weight (similar to hardwood), corrosion proof, fire and termite resistant and very durable. The standard deck panel is 1200mm wide and has a thickness of 30mm. The panel is produced using a continuous process and can be cut to any length. The panels can be cut and drilled using standard wood working practices. A major advantage of the novel sandwich panel technology is its environmental sustainability. It uses patented plant-based polymer technology (approximately 50% bio-content) that permanently stores large volumes of atmospheric carbon resulting in the panel to have low embodied energy. Generally they only use 1/7th of the energy required by conventional materials such as concrete, steel and aluminium.

Figure 1 The composite sandwich panel

CASE STUDIES USING SANDWICH COMPOSITES

FRP sandwich panels have been utilised effectively in Australia in diverse applications. In this section, selected case studies are presented wherein these panels have been engineered to meet the needs of civil infrastructure.

Deck Panels for Small Bridges

Pedestrian bridges and walkway structures with fibre composites sandwich bridge decks (Figure 2) are now common all across Australia. Due to their unique characteristics to withstand the harshest environments while providing a low maintenance, fibre composite sandwich structure in combination with pultruded FRP sections are now the preferred materials in the construction of bridges and walkways near to costal, marine, and environmentally sensitive areas such as tidal flood plains, protected mangrove swamps and corrosive mining facilities. The fabricated deck sections utilizing the novel sandwich panel allow assembly of bridges to be undertaken off site in the factory and then transported to site for very quick installation. This was extremely important where the bridges spanned busy roads that would have required prolonged closure to allow traditional construction methods. Similarly, the sandwich panel can be glued together in several layers to support different loading conditions and for decking solutions that require long spans.

Composite Bridge Beams

Timber bridges are a significant asset in Queensland’s bridge inventory. The task of managing these bridges is becoming increasingly challenging with an ageing timber bridge stock, depletion of the supply of suitable replacement timber elements combined with escalating prices, increasing traffic loads and limited financial resources. In order to address these urgent and complex issues, a collaborative research project was initiated in 2006 to develop commercially viable fibre composite bridge beams (Figure 3). The composite bridge beam incorporates the novel sandwich panel to provide the general shape, shear strength and structural core of the girder while the hybrid modules provide additional flexural strength and stiffness. The steel reinforcing bars of the hybrid elements ensure that the beams have a ductile failure mode which is highly desirable in bridge girders. Selected
cutting and drilling is permitted to ensure the girders are ‘sized’ appropriately for the location in which they are to be installed.

Figure 2 Completed structures with sandwich panel decking

Figure 3 Composite sandwich bridge beams

Transoms

The CFM at USQ, in collaboration with the different railway industries in Australia, has been involved in a number of research and development projects involving innovative composite railway sleepers. These sleepers were developed for areas where railway owners continue to use large numbers of timber sleepers for maintenance including existing timber lines, turnouts and transoms.

Transoms (bridge ties) are large sleepers used on railway bridges to transfer the loads from the rails to the bridge girders. In Australia the rails and the support beams are generally off-set by approximately 250mm which creates significant bending moments and shear forces in the transom. For this type of application, a hybrid polymer sleeper which has sufficient bending and shear strength as well as exhibits a ductile behaviour is developed using the novel sandwich panel as the main structural components. This is achieved by including some steel reinforcement bars at strategically chosen locations within the sleeper. This does not affect the electrical insulation properties of the sleeper and creates a very reliable structural element that gives ample warning of failure. Figure 4 shows the typical cross section of the composite railway transom (Figure 4) wherein the FRP sandwich panels were combined with steel bars similar to the bridge beam concept in Figure 3. In November 2007 the Australian Rail Track Corporation (ARTC) installed twenty two of these transoms on a railway bridge in the Hunter Valley, Australia and to date they have been performing extremely well.

Turnout Sleepers

Turnout sleepers (switch ties) are special sleepers installed in a railway turnout and with varying lengths (up to 4-5m) and fastening locations (Manalo and Aravinthan 2012). A railway turnout enables a train to be guided from one track to another. Because of diminishing availability of long hardwood sleepers, composite turnout sleepers are now being developed to provide an alternative solution to replace deteriorating timber turnout sleepers. This new reinforced polymer sleeper is manufactured using a proprietary casting technique. The sleeper has a prismatic rectangular shape (Figure 5 – left) and contains long glass reinforcement fibres along the length and depth of the sleepers. By strategically concentrating the fibres in locations where they are most effective in carrying the bending moment and shear forces, the total amount of fibres used is significantly less. This economical use of materials
combined with the fast production cycle has resulted in a sleeper which is significantly more cost effective than the alternatives.

Timber Replacement Sleepers

The timber replacement sleepers was specifically designed to conform to the loading conditions for mainline applications in which the sleeper is only loaded in two distinct locations (at the rails) and does not need the same strength along its length. This composite sleeper requires significantly less volume of polymer material while still complying with all strength and stiffness requirements of a timber sleeper is developed and investigated (Figure 5 - right). The shape of the composite sleeper uses only 1/3 the material of a standard rectangular sleeper resulting in a polymer sleeper which is only marginally more expensive than a timber sleeper. The sleepers can be drilled on site similar to timber sleepers, and are available with resilient fasteners or standard rail screws. The innovative casting process creates a very strong layer in the rail seat area and allows for this area to be shaped similar to concrete sleepers which remove the need for steel rail plates. In addition to reducing the amount of polymer material in the sleeper, the novel shape also offers significantly increased resistance against lateral movement (particularly important in curved track). This new sleeper is currently being trialed in Australia by Queensland Rail and in the USA by Union Pacific.

Housing and Construction Applications

The novel sandwich panel has proven to have mechanical properties suitable for housing and construction. In the construction industry the panel has major advantages in wet areas and balcony construction (Figure 6 - left). The panel itself has been classified as a Grade 1 waterproof membrane thus, tiling can start within hours of installation. Recent fire testing has given the panel the top Class 1 rating (Building Code of Australia) which means that the sandwich panel can also be used for walls, roofs, floors and fire doors.

Sandwich Waler

On the coastline of Australia, boardwalks, jetties, pontoons, and marine structures operate in a very corrosive environment. This results in serious durability problems for steel and reinforced concrete. Hardwood has traditionally been used to overcome some of these problems. However, when exposed to aggressive marine environments, timber waler would require replacement every 10 to 15 years. The composite waler made from glue-laminated sandwich structure is a viable substitute for this application because of its excellent corrosion resistance.
and durability properties. The flexural test (Figure 6 - right) indicated that the strength and stiffness of a sandwich waler are suitable for this application. The presence of vertical fibre composite skins in the sandwich waler resulted in a higher resistance to mechanical connections than that of hardwood timber.

Figure 6 Sandwich panel for flooring (left) and composite waler (right)

CHALLENGES AND FUTURE DEVELOPMENTS

A number of issues contributing to the slow uptake of composite sandwich structures in civil infrastructure have been well documented. These important aspects have to be addressed in order to advance their use of sandwich structures in civil engineering applications.

Core Material Development

The core of the sandwich structures for building and construction needs to be reasonably strong to withstand high concentrated and impact loads. Moreover, a thicker composite sandwich panel is usually used in structural than in industrial applications where the shear strength of the core is a critical parameter to efficiently transfer the shear between the top and bottom skins. In fact, Styles et al. (2007) indicated that the shear cracking of the core is the dominant failure mode for a sandwich structure with a thick core. Thus, it is anticipated that the evolution of composite sandwich structure with lightweight, high-strength core and with good capacity for mechanical connections could provide wider opportunity to increase the acceptance and utilisation of this type of construction in civil infrastructure. However, the method of enhancement of the core structure should not involve a complex manufacturing process and increase the cost of production.

Innovative Applications

The growth of composite sandwich construction in civil infrastructure can be further realised by developing innovative structures which exploit its many advantages. Fibre composite sandwich structures will generally feasible in infrastructure when the need for corrosion resistance, high strength, reduced weight, or fast installation is a driver for the system. As seen by the recent development of innovative composite sandwich bridge decks, this effort is driven by the need to replace the heavy weight and corrosion prone reinforced concrete decks and the opportunity to upgrade the load carrying capacity of the existing bridge. Such composite structures are needed in order to address the need in the construction industry for more durable and cost-effective infrastructure. Moreover, typical infrastructure prototypes need to be developed to demonstrate its practical application, increase its acceptance and to build a market volume.

Design Guidelines

The limited understanding by engineers of the overall behaviour of composite sandwich structures is commonly claimed to place them at a disadvantage when considered against traditional construction and building materials. This problem, combined with the lack appropriate design codes and standards, is recognised as a significant barrier to broad utilisation of these materials in civil engineering. Bakis et al. (2002) pointed out that without an established design method and data, it is unlikely that structures utilising fibre composites will be used beyond the scope of research and demonstration projects. Thus, design guidelines for composite sandwich panels should therefore be developed so that it could gain wider acceptance in civil infrastructure.

Structural Health Monitoring

Primary load-bearing FRP sandwich systems are a relatively new technology in civil engineering applications. As a result, their performance history is relatively short compared to more conventional construction systems utilising hardwood, concrete or steel. Thus, short and long-term investigations of their structural (health and) behaviour are essential to develop the market and increase confidence in using these new technologies. The information that will
be obtained from this long-term monitoring and performance evaluation would be helpful in developing design specifications for FRP sandwich systems in civil engineering and construction.

**Education and Training for Civil Engineers**

Several successful R&D projects completed leading to several applications are a testimony that FRP sandwich composites are viable alternative in civil infrastructure. However, there is still a lack of practicing engineers who are trained to design and use fibre composite materials. To fill this gap, courses in fibre composites needs to be developed. USQ has taken the initial steps by developing the first online course on engineered fibre composites with the focus on civil and structural engineers (Aravinthan 2012). This provides a flexible learning environment for practicing engineers who can continue to develop their professional skills in FRP sandwich technologies. Such education and training, especially the young engineers will be a key factor to embrace sandwich composites as an alternative system in civil infrastructure.

**CONCLUSIONS**

This paper has presented recent developments and applications of sandwich composites into civil infrastructure in Australia. Based on this experiences, the following conclusions can be drawn:

- A novel sandwich panel comprises a proprietary and high strength phenolic core material sandwiched between two high strength glass fibre reinforced skins was developed as an answer to minimize the carbon footprint of the construction industry.
- Composite sandwich structure have numerous potential advantages in structural engineering and construction such as high strength, better quality control, easier and faster to install, and reduced energy in transportation.
- Several new and innovative applications have shown that the construction of a more durable and cost-effective infrastructure is possible using FRP sandwich structures. FRP sandwich systems have been successfully used for bridge decks, bridge beams, railway sleepers, and floating structures.

A number of barriers that need to be overcome for the continued growth and wide acceptance FRP sandwich structures are also presented and potential solutions have been identified for this advanced composites technology to become more competitive construction material and harness its potential in civil infrastructure.

**ACKNOWLEDGMENTS**

The authors would like to acknowledge all the industry partners who have been involved in the research and development of diverse application of this technology that has paved way the research and development on the sandwich panel to real-life applications in civil infrastructure.

**REFERENCES**


BEHAVIOUR OF STRUCTURAL SANDWICH COMPOSITES WITH A PHENOLIC CORE AT ELEVATED TEMPERATURE

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ABSTRACT

This paper presents the results of an experimental investigation on the flexural behaviour of sandwich composites under elevated temperature. Sandwich beams made up of glass fiber reinforced polymer (GFRP) composite skins and solid phenolic foam core were tested under 3-point static bending at room temperature and up to 180°C. The results showed that behaviour of the sandwich beams is governed by the GFRP skin up to 80°C while the phenolic core played a major part on the overall behaviour beyond this temperature. The failure of the sandwich beams is initiated by compressive failure of the top skin up to 80°C, while cohesive bond failure and disintegration of the core was observed at higher temperature. Besides, the sandwich beam retained more than 80% of their strength and stiffness at 80°C and this decreases to only 19% at 180°C. Based on these results, the composite sandwich beams can fulfil requirements for civil infrastructure exposed to a service temperature of up to 80°C.

KEYWORDS

Sandwich composites, temperature, phenolic core, flexure.

INTRODUCTION

Fiber reinforced-polymer (FRP) sandwich systems have been successfully applied in civil engineering and construction including structural roofs, walls, bridge beams, bridge decks, and railway sleepers. Still, engineers and asset owners remain cautious about accepting them in civil infrastructure out of concern about its structural performance in applications exposed to elevated temperatures. For example, Sirimanna et al. (2011) measured temperature as high as 61°C for composite sandwich bridge decks exposed to Australian weather conditions. In such conditions, the main concern is the reduction of the sandwich composite’s mechanical load-bearing capacity due to material degradation. Taher et al. (2013) mentioned that the main reason for this concern is the limited and incomplete information about the temperature dependence of sandwich composites. There is a crucial need for research in this area if FRP sandwich systems are to gain acceptance and be used safely for mainstream applications.

To date, there has been limited research and development activities investigating the effect of elevated temperatures on the structural behavior of FRP sandwich composites. Moreover, temperature-related research on polymeric foam cores normally used in FRP sandwich systems is limited as such systems are relatively new in civil engineering applications. Zhang et al. (2015) were probably the first to analytically and experimentally study the influence of elevated temperature on the stability of foam-cored sandwich structures, although the temperature range was only from room temperature up to 90°C. Moreover, skin temperature receives scant attention in all of the existing studies despite the fact that it is significant when the FRP sandwich structure is exposed to elevated temperatures.

Recently, a sandwich composite consisting of glass fiber reinforced polymer (GFRP) composite skins and a solid phenolic foam core has been developed. Extensive characterization of the mechanical properties of this sandwich (Manalo et al. 2010; Manalo 2013) showed that they have strength and stiffness suitable for civil engineering and construction. While the phenolic foam is known to have excellent fire resistance, flame resistance, and high thermal stability (Song et al. 2014) and the GFRP composites have high mechanical properties, no studies have shown that the mechanical properties of a sandwich composite system made from these materials will achieve the required structural performance at elevated in-service temperatures. This paper presents the results of an experimental
investigation of the behavior of this new FRP sandwich system when subjected to the combined simultaneous action of thermal and mechanical loads.

**EXPERIMENTAL PROGRAM**

**Material Properties**

The sandwich panel was manufactured in a single continuous operation, which consists of sandwiching a phenolic-based core between two high-strength GFRP skins. The GFRP skins were comprised of 2 plies of bi-axial (0/90) stitched E-CR glass fabric with a chopped strand mat infused with toughened phenol-formaldehyde resin and had a fiber fraction of 45% by weight. The phenolic core material was a toughened phenol-formaldehyde resin of proprietary formulation owned by LOC Composites Pty Ltd (Australia). Based on the results of the dynamic mechanical analysis (DMA) test, the $T_g$ values measured at the onset of the storage modulus curves are 93°C and 95°C for the GFRP skin and phenolic core, respectively. The mechanical properties of the GFRP skins and the phenolic core are reported in Table 1.

<table>
<thead>
<tr>
<th>Property</th>
<th>Skin</th>
<th>Core</th>
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<tr>
<td>Density (kg/m$^3$)</td>
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<td>855</td>
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<tr>
<td>Modulus of Elasticity, MOE (GPa)</td>
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<td>246.80</td>
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<tr>
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<tr>
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<tr>
<td>Poisson’s ratio</td>
<td>0.25</td>
<td>0.29</td>
</tr>
<tr>
<td>Glass transition temperature, $T_g$ (°C)</td>
<td>93</td>
<td>95</td>
</tr>
</tbody>
</table>

**Specimen Details**

The beam specimens (50 mm in width and 220 mm in length) were cut directly from the manufactured sandwich panels. The top and bottom skins had a nominal thickness of 2.5 mm, while the core thickness was 15 mm (Fig. 1a). Although the beam length was limited by the temperature chamber’s internal clearance, the span of 200 mm (shear span-to-depth ratio of 5) was adequate to initiate skin flexural failure (Manalo 2013). Five replicates for each level of temperature were prepared and tested.

**Test Set-up**

The flexural behavior of the sandwich beams was evaluated with a 3-point static bending test (Figs. 1b and 1c) according to ASTM C393 (2000). Nine temperature cases (RT, 35°C, 50°C, 65°C, 80°C, 100°C, 120°C, 150°C, and 180°C) were considered, where RT represents the control specimens tested at room temperature or at 21°C. The required elevated temperature was achieved with an Instron 3119 environmental chamber mounted on a 100kN servo-hydraulic MTS machine. Prior to testing, the specimens were placed in the environmental chamber at the required test temperature setting for 30 minutes. In addition to the soaking period of 30 minutes in the environmental chamber, the sandwich beams are conditioned in an oven set at the desired temperature while the testing of specimens for a lower test temperature is being conducted. A set of 5 specimens were tested for each temperature range and were loaded at a rate of 3 mm/min.
RESULTS AND DISCUSSIONS

Load-deflection Behavior

Figure 2 shows the typical load–deflection behavior of the sandwich beams at elevated temperature. As can be seen from the graph, almost linear load–deflection behavior was observed up to failure for sandwich beams tested at RT and up to 80°C. At 100°C to 180°C, a reduction in the stiffness and failure load was observed. The load–deflection behavior of the sandwich beams tested under these temperatures is characterized by an initial linear elastic phase, followed by a nonlinear phase until a maximum value was reached. After this, only a small increase in load but with a significant increase in deflection until failure occurred. The nonlinear behavior of these beams resulted from the low, steady decrease in the phenolic core’s mechanical properties between these temperature ranges. Garrido et al. (2015) observed similar behavior for PET foam, which exhibited reduced stiffness and increasing nonlinear behavior at higher temperatures. The significantly low load capacity of the sandwich beams at 150°C and 180°C is due to the strong degradation of the phenolic core. Surprisingly, a slightly higher slope at the linear elastic portion of the load-deflection curve was observed for sandwich beams tested at 180°C than 150°C. This can be due to the evaporation of moisture in the matrix, making the laminate brittle and regaining some of its stiffness. Lauobi et al. (2014) reported a similar observation and evaporation of moisture from glass-fiber unsaturated polymer composites at higher temperature.

Failure Behavior

Figure 3 shows the typical failure behavior of the sandwich beams at elevated temperature. Compressive failure of the top skin was observed in the sandwich beams tested at RT and up to 80°C. This was followed immediately by core shear and interlaminar shear failure of the bottom GFRP skin, which extended up to the edge of the beam. Figures 3a and 3b clearly show the delamination between the plies of bottom skin and fibers still attached to the phenolic core. The initiation of compressive failure of the skins was also observed in the sandwich beams tested at 100°C and 120°C. The core experienced cohesive bond failure, followed by more progressive failure behavior than in the specimens tested at up to 80°C. This failure was confined to the area between the loading point and support, as shown in Figure 3c. No loss of bond was observed in the specimens tested at 150°C and 180°C, but the core disintegrated, resulting in shear failure. The disintegration of the core is characterized by micro cracks along the length of the beam which eventually grow in number and start to interact with each other forming a shear crack. Core indentation occurred under the loading point, which clearly indicates softening of the phenolic core at elevated temperature. These observed failure behaviors differ from those observed by Goodrich and Lattimer (2012), in which they indicated that sandwich composites with foam cores at elevated temperature normally fail.
by face-sheet debonding. The observed failure of sandwich panels in this study suggests the high quality of bond between the GFRP skins and phenolic core, thereby preventing debonding failure of the top and bottom skins.

Figure 3 Failure behaviour of the sandwich beams at elevated temperature

(a) RT and 35°C
(b) 50°C to 80°C
(c) 100°C and 120°C
(d) 150°C and 180°C

Effect of Elevated Temperature on Sandwich Beams’ Properties

Figure 4a shows the maximum bending stress (BS) and the overall bending stiffness (OBS) of the sandwich beams. When the maximum BS and OBS were calculated, the sandwich beams were assumed to have a homogenous rectangular cross section. It is important to note that shear deformation of the phenolic core was not considered in the analysis of the sandwich beam. This assumption is reasonable as it was found in the earlier investigation that the behavior of this sandwich beam with an a/d of 5 is governed by flexure and with only minor contribution by shear (Manalo 2013). Moreover, the results of the investigation by Manalo et al. (2010) showed that the contribution of shear in the total deflection of the sandwich beam is only around 5% due to the high shear modulus and strength of the phenolic core.

It can be clearly see from Figure 4a that both the BS and OBS of the sandwich beams decreased with increasing temperature. The maximum BS was affected more by the high temperature than the OBS. This is because the OBS was measured at the linear region of the load–deflection curve, while the BM was based on the point of maximum load. Moreover, the thermal degradation of the phenolic core resulted in reduced load transfer between the top and bottom skins. It can also be seen that there were no significant changes in the sandwich beam’s OBS up to 80°C. In this case, the value of 1.11 kN/mm at RT dropped only to 1.03 kN/mm at 80°C. The OBS decreased rapidly at temperatures in excess of 80°C. The strength of almost 135 MPa decreased to 99 MPa at 80°C and to only 25 MPa at 180°C. In the case of sandwich beams under bending, the top and bottom skins are subjected to compressive and tensile stresses, respectively, while the core resists shear. Gu and Asaro (2009) indicated that sandwich composites will experience a significant reduction in strength and stiffness once the skin has completely degraded due to the core's low bending stiffness. This study determined that the GFRP skins contributed significantly to the overall flexural behavior of the sandwich beams up 80°C, but the phenolic core played a major role at higher temperatures. Zhang et al. (2015) indicated that this is because the shear modulus of foam-core materials decreases at elevated temperature.

The OBS of the sandwich beam [SW (OBS)] at different levels of temperature under 3-point static bending was normalized by dividing the values by the overall bending stiffness of the specimens obtained at room temperature (Figure 4b). In this figure, the normalized storage moduli of the skin [Skin (DMA)] and the phenolic core [Core

1315
(DMA)] from the DMA tests was also reported for comparison. It can be seen from that the normalized bending stiffness of the SW (OBS) and the Skin (DMA) followed the same trend up to 120°C, but began deviating beyond this temperature. The increase in the storage modulus of the Skin (DMA) at a temperature around 130°C is due to the evaporation of moisture in the matrix, making the laminate brittle. The decreasing effective bending stiffness of the sandwich beam with increasing temperature is due to the phenolic core’s thermal degradation. Still, SW retained 84% of its stiffness at 100°C and 69% at 120°C. In comparison, Zhang et al. (2015) reported that only about 5% of the bending stiffness and 8% of the yield strength were lost with a sandwich beam with a PVC foam core at 90°C. This is expected as the aluminum alloy used for the face sheet had high heat conductivity, and its stiffness and strength were not affected up to 90°C. This indicates that the strength of sandwich beams at temperatures lower than the $T_g$ of the constituent materials will be governed by the GFRP skin’s strength. This is supported by the observed failure behavior, in which the the sandwich beam failed due to compressive failure of the top skin, followed by core shear. Compressive rather than tensile failure was expected as the GFRP skin had greater tensile than compressive strength, as shown in Table 1. Similarly, Feih et al. (2007) indicated that FRPs were more stable in tension than in compression at high temperature. In their study, they observed that the compressive strength of woven glass/vinyl ester laminate decreased by 80% when subjected to temperatures between 50°C and 120°C, but that the tensile strength only decreased by 15%. The large discrepancy between the behavior of sandwich beam and the skin at 150°C and 180°C could be attributed to the disintegration of the phenolic core, as shown in Figure 3d, thereby reducing the load transfer between the top and bottom skins.

CONCLUSIONS

This study investigated the flexural behavior of sandwich composites consisting of GFRP skins and a phenolic core at elevated in-service temperature. Based on the results of this study, the following conclusions have been drawn:

- The bending strength and stiffness of the sandwich beams decreased with increasing temperature, with the bending strength was affected more than the stiffness.
- The flexural behavior of the sandwich beams is governed by the GFRP skin up to 80°C, while the phenolic core played a major part in the overall behavior beyond this temperature. The failure of the sandwich beams was initiated by the compressive failure of the skin up to 80°C, while cohesive bond failure and disintegration of the phenolic core were observed at higher temperatures.
- The sandwich beam retained more than 80% of their strength and stiffness at 80°C and this decreases to only 19% at 180°C. Based on these results, the composite sandwich beams can fulfil requirements for civil infrastructure exposed to a service temperature of up to 80°C. This indicates that FRP sandwich systems made from these materials can be used for civil-engineering structures with in-service temperatures of up to 70°C.
- The full composite action between the skins and the core was lost due to the disintegration of the phenolic core at higher temperatures, which should be accounted in the prediction of the strength and stiffness of the sandwich beams under elevated temperature.

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STRUCTURAL CONCEPT AND DESIGN OF A GFRP-POLYURETHANE SANDWICH ROOF STRUCTURE

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ABSTRACT

This work presents the concept and structural design of a 1000-m² glass fiber-reinforced polymer (GFRP) sandwich roof replacing a timber roof over an indoor swimming pool. The advantages offered by the lightweight, multifunctional GFRP-polyurethane (PUR) sandwich construction for the roof replacement are described. The roof structure was designed for two potential construction scenarios (with and without including a green rooftop), whose pertinent dead loads on the roof significantly differed. The resulting designs for both cases, considering long-term effects and applying two design recommendations (Eurocomp and BÜV) are compared and discussed.

KEYWORDS

Sandwich structures, roof, GFRP, polyurethane, structural design.

INTRODUCTION

Within the framework of renovation works, the existing timber roof of the CLP building in Switzerland is replaced by a glass fiber-reinforced polymer (GFRP) sandwich roof. The top floor of the CLP building accommodates two main uses, namely an indoor swimming pool and a reception zone (see plan view in Figure 1(a)). The almost entire top floor is currently covered by an approximately 1000-m² area (up to 54-m length by up to 23-m width) flat green timber roof with triangular openings for zenithal daylighting. Based on architectural and aesthetic considerations, the new roof aims to provide the swimming bath’s inside space with architectural spatial effects while maintaining illumination through natural light and virtually preserving the nearly trapezoidal perimeter geometry of the original timber roof. Furthermore, the new roof structure has to be borne by the underlying vertical structure within its limited load-bearing capacity, therefore preventing the need for structural reinforcement. The existing structure consists of prefabricated steel/concrete columns around the swimming pool and along the North and East facades delimiting the reception zone, as well of reinforced concrete core walls which allocate the technical building equipment and create the spatial division between the two building uses, see Figure 1(a). Besides, demanding serviceability and durability requirements, in particular the high operating temperature and relative humidity in the swimming pool environment, had to be met.

Figure 1 (a) Plan view of CLP Building, dimensions in [m]; (b) panel arrangement of CLP sandwich roof

Following the architectural concept, double-curved enclosure shapes enabling the introduction of natural light across the arising openings are designed for the swimming pool area, as shown in Figure 2. On the other hand, for
the lobby area, with lower aesthetic requirements, a flat roof solution is used. Considering the double-curved geometry, the roof is conceived in lightweight GFRP sandwich construction, which allows both for complying with the aforementioned constraints and building the designed complex shapes. Moreover, the merging of aesthetic architectural functions (roof shape, daylighting and surface appearance), static functions (vertical load transfer and bracing of the existing base-hinged columns) and building physical functions (thermal insulation provided by the selected core material) into the sandwich single components will contribute to reduce and accelerate on-site works, thus shortening the closure time of the building.

![Figure 2 Rendering of CLP sandwich roof (a) exterior and (b) interior views](image)

The roof is envisaged to be prefabricated using the vacuum assisted resin infusion process in the form of ten primary components, see Figure 1(b). Each wave form is comprised within one panel, and the impact of the complex shapes on the manufacturing cost is decreased by the repeated use of one (expensive) mold for elements 3–5 in Figure 1(b). The resulting panels, generally 5.00-m-wide and with a length equal to the roof’s width, are transportable by road to the site, and their installation can be done almost simultaneously with the dismantling of the existing timber roof. Worksite construction comprises the execution of the panel-to-panel joints, which are vacuum infused. In order to fulfill urban planning specifications, the installation of a green rooftop may be required; an extensive sedum roof (shallower and lighter) is selected against an intensive green roof.

### STRUCTURAL CONCEPT AND MATERIALS

The plan view of the roof structure, indicating the supports location, and a longitudinal cross section are shown in Figure 3. The geometry of the sandwich roof consists of three identical double-curved wave shapes with a 1.25-m rise and 8.80-m span above the swimming bath (see axes D–F in Figure 3). The roof is nearly flat elsewhere and has pre-cambered, 1.70-m-long wing-shaped overhangs along the perimeter facades. The sandwich structure has a variable 300–600 mm thickness (tapering off to 90 mm at the overhangs ends) so as to guarantee the minimum slope required for rainwater evacuation. This option was selected against an inclined roof of constant thickness due to two reasons: (i) to keep horizontal the roof’s bottom surface – this was essential to simplify the use of the existing underlying structure as supports; (ii) to have horizontal cornice lines. The roof structure mainly spans transversely to the building’s longitudinal direction, i.e., in the East-West or X direction, with an 8.80-m maximum span located over the swimming bath and given by the spacing of the existing columns around it (see axes 2-3 in Figure 3(b)). The transverse span (in the North-South or Y direction) is of 5.00 m and determines the roof division for manufacturing purposes – panels 2–8 in Figure 1(b) have a constant 5.00-m width.

The sandwich structure is composed of GFRP face sheets and a polyurethane (PUR) foam core. The GFRP face sheets consist of E-glass fibers and a fire- and flame-retardant epoxy resin. Based on the manufacturing process, a total fiber volume fraction of 55% is considered. Unbalanced cross-ply laminates, with 2/3 and 1/3 of the total UD layers directed in the main (X) and transverse (Y) span directions, were selected. The core globally consists of a PUR foam of 60-kg/m³ density, which also provides the thermal insulation required for building physical performance. The PUR foam core is locally reinforced by vertical GFRP webs or ribs. In each sandwich panel, one internal main web (beam) running parallel to the main span direction and aligned with the supports contributes to increase the shear stiffness and strength of the non-reinforced PUR foam core. Internal secondary webs are added in the transverse span direction, crossing the main ones at the supports, to avoid potential punching failure of the sandwich structure at the location of the concentrated reaction forces. Besides, external webs laterally envelope the panels, conforming double webs once adjacent panels are connected. The arrangement of webs is depicted in Figure 3(b). The same total fiber volume fraction as for the face sheets is used for the webs; however, balanced cross-ply laminates, with 1/2 and 1/2 of the fibers directed along the web’s length and height, respectively, are selected instead. The main mechanical properties of the GFRP laminates, both for the face sheets and for the webs, and of the PUR foam are listed in Tables 1 and 2.
Table 1 Properties of GFRP laminates, average values calculated based on selected fiber content and architecture

<table>
<thead>
<tr>
<th>Property</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus</td>
<td>$E_{Lx,m}$</td>
<td>29 GPa</td>
</tr>
<tr>
<td></td>
<td>$E_{Ly,m}$</td>
<td>18 GPa</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>$G_{Lx,m}$</td>
<td>2.4 GPa</td>
</tr>
</tbody>
</table>

Table 2 Properties of PUR foam, characteristic values from Keller et al. (2008)

<table>
<thead>
<tr>
<th>Property</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear strength</td>
<td>$\tau_{C,k}$</td>
<td>0.25 MPa</td>
</tr>
<tr>
<td>Elastic modulus</td>
<td>$E_{C,k}$</td>
<td>17.5 MPa</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>$G_{C,k}$</td>
<td>6.5 MPa</td>
</tr>
</tbody>
</table>

STRUCTURAL DESIGN

The sandwich roof was designed for two potential construction scenarios: (i) A roof: the sandwich finishing surface provides the roof’s exterior appearance, i.e., no additional construction elements are added above; (ii) B roof: a sedum green roof of 170-kg/m$^2$-weight is installed on the top. The structural design was carried out according to the requirements of Swiss standards (SIA), namely SIA260 (2013) and SIA261 (2014), which are consistent with the corresponding Eurocodes. The partial safety factor concept, with partial factors applied both on the action and material sides (load and resistance factors, respectively), was used. The following actions were considered: permanent loads (self-weight of the sandwich structure and roof system) and snow; their associated load factors were determined from SIA260 (2013). Material resistance factors were obtained from Eurocomp (Clarke 1996) and from the German BÜV (2014), both in line with the Eurocodes philosophy, as neither the Swiss standards nor the Eurocodes provide such values for FRP and PUR foam materials.

The preliminary structural design, both for Eurocomp and BÜV design recommendations, was conducted based on the selected materials and the basic module depicted in Figure 4, representative of the sandwich roof structure. Only the design of the “beam” and “slab” elements are referred here. The longitudinal main web conjointly with 1.0-m width from the upper and bottom face sheets was considered as a wide flange beam, designated as “beam” in the following. The web and face sheet thicknesses were determined to verify the ultimate (ULS) and serviceability (SLS) limit states for the beam element, respectively. The resulting face sheet thickness were afterwards used to verify the slab’s ULS and SLS (constant thickness laminates are used as face sheets).
The resistance factor for FRP materials according to Eurocomp, $\gamma_m$, comprises at ULS three sub-factors ($\gamma_m = \gamma_m^1 \cdot \gamma_m^2 \cdot \gamma_m^3$) relating to the property source, manufacturing process, operating temperature and load duration. These factors are applied to the characteristic (5%-fractile) strength and stiffness values of the laminate and core properties to obtain the design values. The characteristic values can be obtained reducing by 0.8 the relevant average values (DIN 18820-2). For the verification of the SLS, the average stiffness properties are reduced by $\gamma_m = 1.30$ and creep is considered by a further stiffness reduction factor, $\chi$ (see Figure 4.13 in Eurocomp). These factors were also used for the PUR foam. In BÜV, the resistance factor, $\gamma_m$, depends on the design situation, material type (laminate, resin or foam) and manufacturing method. A material-specific value is formed increasing $\gamma_m$, both at ULS and SLS, by an additional modification factor, $A_{\text{mod}} \geq 1$; this is built-up from three sub-factors ($A_{\text{mod}} = A_1 \cdot A_2 \cdot A_3$) accounting for the load duration, environmental exposure and ambient temperature. Different $A_{\text{mod}}$ factors apply for strength (superscript $f$) and stiffness (superscripts $E$ or $G$). The two sets of factors used according to Eurocomp and BÜV are listed in Tables 3 and 4, respectively.

### Table 3 Resistance factors, sub-factors and stiffness reductions according to Eurocomp used in CLP roof design

<table>
<thead>
<tr>
<th>Material</th>
<th>Element</th>
<th>Time scenario</th>
<th>$\gamma_m^1$</th>
<th>$\gamma_m^2$</th>
<th>$\gamma_m^3$</th>
<th>$\gamma_m$</th>
<th>$\gamma_m \chi$</th>
<th>$\gamma_m \chi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>GFRP</td>
<td>Face sheets, webs</td>
<td>Short-term $^{(1)}$</td>
<td>1.50</td>
<td>1.20</td>
<td>1.20</td>
<td>2.16</td>
<td>1.30</td>
<td>0.79</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Long-term $^{(2)}$</td>
<td>1.50</td>
<td>1.20</td>
<td>3.00</td>
<td>5.40</td>
<td>1.30</td>
<td>0.57</td>
</tr>
<tr>
<td>PUR foam</td>
<td>Core</td>
<td>Short-term $^{(1)}$</td>
<td>1.50</td>
<td>1.20</td>
<td>1.20</td>
<td>2.16</td>
<td>1.30</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Long-term $^{(2)}$</td>
<td>1.50</td>
<td>1.20</td>
<td>3.00</td>
<td>5.40</td>
<td>1.30</td>
<td>0.27</td>
</tr>
</tbody>
</table>

1. week; 2.50 years; 3. intermediate value for properties derived from theory/tests; 4. vacuum infusion, fully post-cured; 5. operating design temperature 25–50°C; 6. CSM/WR preferred line; 7. UD shear

### Table 4 Resistance factors and modification factors according to BÜV used in CLP roof design

<table>
<thead>
<tr>
<th>Material</th>
<th>Element</th>
<th>Time scenario</th>
<th>$\gamma_m$</th>
<th>$A_{\text{mod}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>GFRP</td>
<td>Face sheets, webs</td>
<td>Short-term $^{(1)}$</td>
<td>1.35</td>
<td>1.37</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Long-term $^{(2)}$</td>
<td>1.35</td>
<td>1.49</td>
</tr>
<tr>
<td>PUR foam</td>
<td>Core</td>
<td>Short-term $^{(1)}$</td>
<td>1.50</td>
<td>3.07</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Long-term $^{(2)}$</td>
<td>1.50</td>
<td>5.79</td>
</tr>
</tbody>
</table>

Design of Webs

The web thickness, $t_w$, was designed to verify ULS of shear and shear wrinkling, both for short- (variable and permanent) and long-term (permanent) loading. Shear buckling stress, providing a low bound for shear strength and therefore a $t_w$ value on the safe side, was also verified at long-term (i.e. the foam core, whose lateral support is reduced with time owing to creep, is neglected). An $h_w = 500 \text{ mm}$ web depth (value at the cross section where...
the shear force is maximum) was considered for all verifications. According to Eurocomp, the shear area, $A_s$, equals the web cross-sectional area ($A_s = t_w \cdot t_e$), whereas in BÜV this value is reduced by 1.5 to take into account the non-uniform distribution of shear stresses. A basic web shear strength of $t_w = 50$ MPa was used for the ULS shear verification. The wrinkling shear strength was verified according to $\tau_{wr,d} = 0.5 \sigma_{wrd}$ (Keller et al. 2008), with the design value of the uniaxial compression wrinkling strength, $\sigma_{wrd}$, given by Eq. (1) (BÜV 2014):

$$\sigma_{wrd} = 0.82 \sqrt{\frac{E_{L,d}}{G_{C,d}}}$$

where $E_{L,d}$ = design value of the laminate elastic modulus, $E_{C,d}$ = design value of the core through-thickness elastic modulus, $G_{C,d}$ = design value of the core through-thickness shear modulus. The critical shear buckling stress was calculated according to Eq. 2 for especially orthotropic materials (Clarke 1996):

$$\tau_{cr,d} = \left[4k \cdot (D_x \cdot D_y)^{1.25}\right]/(h_w^2 \cdot t_w)$$

where $k = 8$; $D_x$ = plate flexural rigidity in the $X$ direction, $D_y$ = plate flexural rigidity in the $Y$ direction. The resistance factors applied for Eurocomp design are identical for the three referred verifications, see Table 3. For BÜV design, resistance and modifications factors relating to local stability apply for $\tau_{wrd}$ and $\tau_{cr,d}$; besides, $\tau_{cr,d}$ depends on three material properties (see Eq. 1) for which different factors apply. As a result, the global $f_{M} \cdot A_{mod}$ values for $\tau_{cr,d}$ differ as follows: 2.08/2.75/1.3 and 2.47/3.54/2.04 for short- and long-term scenarios. Figure 5(a) shows the minimum $t_w$ required for each design case; the resulting thicknesses are encircled (those obtained from the shear buckling verification are not considered). Applying the Eurocomp factor set, the web is designed for shear, due to the slightly higher characteristic shear wrinkling strength ($\tau_{cr,d} = 52$ MPa) compared to the basic shear strength ($t_w = 50$ MPa) and the equal resistance factors applied in both cases at the design level. The long-term scenario is determining, both for $A$ and $B$ roofs, since the resistance factor is 2.5 times higher at long- than at short-term, whereas the ratio of the short-long-term loads is smaller (2.3 / 1.4 for $A$ / $B$ roofs). For the design according to BÜV, the governing design case is shear wrinkling, for which the greatest resistance factors apply and compensate for $\tau_{wr,k} > \tau_{wrd}$. The short-term scenario is decisive, as the aforementioned short-long-term loading ratio is not counteracted by the only 1.3 higher resistance factor at long-term. For $A$ roof, higher $t_w$ was obtained for BÜV (where a lower $A_e$ is considered) than for Eurocomp design (6 vs. 4 mm). On the other hand, similar $t_w$ values (11 mm) were obtained for $B$ roof with both recommendations in spite of the different governing design case.

![Figure 5 Design of beam: (a) web thickness and (b) minimum face sheet thickness](image)

**Design of Face Sheets**

The face sheets thickness, $t_f$, was determined so that the beam element verified the deflection at SLS. Timoshenko’s beam theory, the $t_f$ values obtained from the web’s design and the minimum web’s depth ($h_w = 420$ mm) were used to calculate the deflections. Rotationally free beam end conditions were considered for the short-term loads (assumption on the safe side) and partially restrained ends for the permanent loads (to take continuity into account). The deflection limits according to SIA260 (2013) were span/350 and span/300 for variable and permanent loads including creep, respectively. The minimum $t_f$ required for each design case is shown in Figure 5(b) and the resulting thicknesses are encircled. For $A$ roof, the short-term case is determinant, both for Eurocomp and BÜV designs, due to several reasons: the higher (~1.3 times) value of the variable load compared to the permanent load, the simply-supported end conditions and the more restrictive deflection limit applicable. For the $B$ roof, on the other hand, the significantly higher permanent loads (more than 2.5 times the variable load), resulting in also significantly higher long-term creep deflections, counteract the more favourable end conditions and deflection limits of the short-term verification; consequently, the long-term becomes the governing design case. Higher $t_f$ values were obtained for Eurocomp than for BÜV design (12 / 21 mm vs. 8 / 17 mm for $A$ / $B$ roofs, respectively).
in part due to the more restrictive Eurocomp \( y_w/\chi \) values for the face sheets compared to the pertinent \( y_M/A_{mod} \) values from BÜV. At ULS, the face sheets’ compressive stress was verified against wrinkling strength according to Eq. (1) (the face sheet elastic modulus in the main span direction, \( E_{L,x,d} \), is used). The results are given in Figure 6(a), which shows that the ULS of the face sheets is verified and that they are overdesigned in terms of strength (a maximum stress/strength ratio of 41\% was obtained).

<table>
<thead>
<tr>
<th>Axial stress in face sheet, ( \sigma_{f,d} ) (MPa)</th>
<th>( \sigma_{w,r,d}=32.0 ) MPa</th>
<th>( \sigma_{w,r,d}=41.2 ) MPa</th>
<th>( \sigma_{w,r,d}=21.0 ) MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>A roof:</td>
<td>( \sigma_{f,d}=32.5 ) MPa</td>
<td>( \sigma_{f,d}=41.2 ) MPa</td>
<td>( \sigma_{f,d}=21.0 ) MPa</td>
</tr>
<tr>
<td>Short-term</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Long-term</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Eurocomp BÜV

<table>
<thead>
<tr>
<th></th>
<th>( \sigma_{w,r,d}=44.8 ) MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>A roof:</td>
<td></td>
</tr>
<tr>
<td>Short-term</td>
<td>( \sigma_{f,d}=35.1 ) MPa</td>
</tr>
<tr>
<td>Long-term</td>
<td>( \sigma_{f,d}=27.5 ) MPa</td>
</tr>
<tr>
<td>Short-term</td>
<td>( \sigma_{f,d}=35.1 ) MPa</td>
</tr>
<tr>
<td>Long-term</td>
<td>( \sigma_{f,d}=27.5 ) MPa</td>
</tr>
</tbody>
</table>

Figure 6 Verification at ULS of (a) beam face sheets and (b) sandwich slab

Verifications of Sandwich Slab

ULS verifications of the sandwich slab concerned the shear strength of the core and the wrinkling strength of the compressed face sheets in the transverse (Y) span direction, the latter according to Eq. (1) with \( E_{L,y,d} \) for the laminate elastic modulus. Figure 6(b) summarizes the verifications and shows that ULS is fulfilled for both components: the maximum \( E_d/R_d \) values obtained (ratio of effect of actions to resistance) were \( V_d/V_{R,d} = 79\% \) for the core and \( \sigma_{u,d}/\sigma_{w,r,d} = 6\% \) for the face sheets, which are therefore largely overdesigned for ULS in the Y direction. All the deflection limits were met except for B roof design according to BÜV recommendations, with a resulting long-term deflection of approximately span/250. The addition of an intermediate transverse web parallel to the Y direction and reducing the slab span in X direction from 8.80 to 4.40 m is needed.

CONCLUSIONS

A lightweight sandwich roof consisting of GFRP face sheets and PUR foam core is designed to replace the existing timber roof of the CLP building. The new GFRP-PUR sandwich roof, integrating structural, building physical and architectural functions into large-scale, prefabricated and lightweight elements, allows for using the existing vertical structural members as supports within their limited load-bearing capacity and for reducing installation times. The roof dimensions and composition depend on the selected design recommendations (Eurocomp / BÜV); however, material consumption may be comparable (Eurocomp leads to thicker face sheets, whereas thicker webs or additional intermediate transverse webs are required by BÜV). The considered construction scenario (with / without green rooftop) significantly influences the final design. In particular, the considerably higher (3.4 times) permanent loads of the green roof lead to notably greater thicknesses of the web (1.8–2.8 times) and face sheet (1.8–2.1 times) laminates. As a result, the self-weight of the slab increases by up to 60\%. The substantial influence of high permanent loads on the roof solution highlights the relevance of considering long-term performance and creep effects in the design of FRP sandwich structures.

REFERENCES


Swiss Society of Engineers and Architects (2014). *SIA 261: Actions on structures*, SIA, Zurich, Switzerland.
TWIN-SHAPED CFRP-SANDWICH PEDESTRIAN BRIDGE

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2 University of Applied Sciences and Arts of Southern Switzerland (SUPSI), Manno, Switzerland
3 Ernst Basler + Partner AG, Zurich, Switzerland
4 BASE Srl., Alserio CO, Italy
5 Ecole Polytechnique Fédérale de Lausanne (EPFL), Lausanne, Switzerland

ABSTRACT

A concept of a nature-inspired pedestrian bridge which tries to merge architectural, structural and manufacturing aspects in an optimized way is presented. The 18.0-m-span bridge is designed as an overhead spatial frame of a complex double-curved shape, which is composed of twelve modules consisting of lightweight carbon fiber-reinforced polymer (CFRP) sandwich construction. The modules are fabricated with only one mould through vacuum assisted resin infusion. The aimed visible texture and black colour of the CFRP fabrics represents an essential architectonical element. The resulting benefits and required compromises of this multi-criteria approach are discussed.

KEYWORDS

CFRP, bridge, module, sandwich.

INTRODUCTION

The new 18.0-m-span TSCB (twin-shaped composite beam) pedestrian bridge connects the villages of Bissone and Melide on the eastern and western side of the lake of Lugano in Switzerland. The architectural and structural concept is inspired by natural microorganisms, e.g. radiolarians and diatoms, whose skeletons are structurally optimized lightweight constructions. The resulting bridge structure is conceived as an overhead spatial frame of 2.40-m height, as shown in Figures 1 and 2. The roof frame and walkway slab are in parallel, horizontal planes while the side frames are undulated and form three waves of 6-m length each, see Figures 3 and 4. This undulation results in an also undulating width of the roof frame and walkway slab along the bridge, which are staggered however by one half-wave length, i.e. in the cross sections with the largest width of the walkway slab the width of the roof frame is the smallest and vice versa. The width of the walkway thus varies between 1.36 and 3.36 m. The edges between the horizontal and the undulated side frames are double-curved. Furthermore, transverse stiffeners are located around the frame openings to increase the buckling resistance of the compressed frame struts.

Figure 1 Rendering of TSCB bridge

Following the aim of approaching the efficiency of natural structures, the components of the lightweight spatial frame are conceived in sandwich construction. Carbon fiber-reinforced polymer (CFRP-) materials were selected for the face sheets since the architectural concept aimed to have a black coloured bridge with a surface finish that shows the texture of the fabrics.
The geometry of the bridge was not only a result of the architectural and structural conception, but also optimized from the manufacturing point of view. The overall complex shape is conceived in a way that it can be built up from only one complex module of 3.0-m length, as shown in Figure 5, which requires only one (expensive) mould for manufacturing. The openings in the bottom frame are closed to form the walkway slab. Furthermore, the undulating joints between the upper and lower modules are located in vertical planes to reduce the geometrical complexity, see Figure 4; the overall bridge width is thus constant (4.20 m), see Figure 3. Vacuum assisted resin infusion was selected for the manufacturing with the mould being placed on the inner side to have the smooth surface there. All the joints are vacuum infused or adhesively-bonded in the factory; the whole bridge is thus prefabricated and transported to the site.

MATERIALS AND STRUCTURAL MODEL

The thickness of the roof and side frame sandwiches is 30 mm, and that of the walkway slab 40 mm. The CFRP face sheets are 0.75-mm thick and composed of three layers of fabrics (SAATI CC283, 49/51% in 0/90° direction) and an epoxy resin (Conchem LC 318/IN). Lightweight (280-g/m²) twill-weave fabrics were selected due to the required drapability. The three layers are rotated against each other by 30° (arranged at 0/30/60°) to form quasi-isotropic laminates since an anisotropic lay-up, that would better adapt to the stress fields, was inconceivable considering the complex geometry. The 3-layer laminates are thus asymmetric, the resulting through-thickness stresses in the core are however insignificant. The laminate lay-up is doubled to 6 layers (in a symmetric
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arrangement) in the walkway slab and, in the curved edge regions marked in yellow in Figure 6, it is increased to
6 or 12 layers. The laminates are UV-protected by a transparent coating. The core material is an orthotropic
polyethylene terephthalate (PET) foam of 135-kg/m³ density (Airex T92.130). The foam is placed with the
extrusion direction and weldline planes perpendicular to the face sheets (in "end-grain" direction). The foam is
substituted by an isotropic solid polyurethane (PUR) material (SikaBlock M930) in the support cross beams (blue
region in Figure 6) and the double-curved curved edge regions.

The laminate properties and joint strengths were obtained from testing, the core properties from the manufacturer.
The main properties of the CFRP laminates and PET foam at room temperature are given in Tables 1 and 2. Due
to the black colour, the surface temperature could locally increase up to 84°C, as corresponding measurements on
CFRP sandwich samples protected with a transparent UV-coating and exposed to a normal irradiation of
1000 W/m² (as present in Bissone) have shown. The maximum allowable service temperature, however, was
defined as the onset value of the glass transition temperatures, \( T_{g,\text{onset}} \) (according to dynamic mechanical analysis,
DMA), of the resin or foam minus 10°C. Surfaces which would exceed this temperature are painted with a light
colour to limit the temperature increase to the allowable temperature.

The replacement of the PET foam in the strongly curved edge regions by the solid PUR material was necessary
since the foam could not be placed in end-grain direction. A deviation of this direction, however, was linked with
a drop of the properties due to the material orthotropy. Several core materials were evaluated to solve this problem,
as it is shown by a comparison of some significant properties listed in Table 3. A comparable high-density (almost)
isotropic polyurethane foam (PUR 145) has a better tensile strength than the selected PET foam transverse to its
extrusion direction (index \( \perp \)), however, the properties parallel to the PET foam extrusion direction (index \( \| \)) were
much lower. Orthotropic Balsa wood, on the other hand, has much better properties in grain direction than the PET
foam in extrusion direction. However, the tensile strength in the critical transverse direction is lower. It was thus
not possible to find only one lightweight material (at a reasonable cost) that would fulfil all the requirements and
the selected two-material solution resulted as being the best one.
Table 1 CFRP laminate properties

<table>
<thead>
<tr>
<th>Property</th>
<th>3-layer asymmetric</th>
<th>6-layer symmetric</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Orientation</td>
<td>0/30/60°</td>
<td>0/30/60/30/0°</td>
<td></td>
</tr>
<tr>
<td>Thickness, (t_L) (mm)</td>
<td>0.75</td>
<td>1.50</td>
<td>in 15°-direction</td>
</tr>
<tr>
<td>Tensile strength, (f_{Lt,k}) (MPa) (^1)</td>
<td>290</td>
<td>520</td>
<td></td>
</tr>
<tr>
<td>Elastic tensile modulus, (E_{Lt,k}) (MPa) (^1)</td>
<td>24200</td>
<td>45300</td>
<td>Average value</td>
</tr>
<tr>
<td></td>
<td>31700</td>
<td>51200</td>
<td>0/15°-directions</td>
</tr>
<tr>
<td>Tensile failure strain, (G_{Lt,k}) (%) (^2)</td>
<td>1.6</td>
<td>1.5</td>
<td>Average value</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0/15°-directions</td>
</tr>
<tr>
<td>Poisson ratio, (\nu_L) (-)</td>
<td>0.32</td>
<td>0.32</td>
<td>Calculated value</td>
</tr>
<tr>
<td>Thermal elongation coefficient, (a_{cm}) (K(^{-1})) (^2)</td>
<td>13.6E-6</td>
<td>17.5E-6</td>
<td>in 0°-direction</td>
</tr>
</tbody>
</table>

\(^1\) characteristic, \(^2\) average values

Table 2 PET foam properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Values</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile strength, (f_{Ct,k}) (MPa) (^1)</td>
<td>2.11</td>
<td>Extrusion direction</td>
</tr>
<tr>
<td>Compression strength, (f_{Cc,k}) (MPa) (^1)</td>
<td>2.15</td>
<td>Extrusion direction</td>
</tr>
<tr>
<td>Shear strength, (\tau_{C,k}) (MPa) (^1)</td>
<td>1.14</td>
<td>Extrusion direction</td>
</tr>
<tr>
<td>Elastic modulus, (E_{C,m}) (MPa) (^2)</td>
<td>50</td>
<td>(\perp) to extrusion direction, FEM</td>
</tr>
<tr>
<td>Elastic modulus, tension, (E_{Ct,k}) (MPa) (^1)</td>
<td>138</td>
<td>Extrusion direction, wrinkling verification</td>
</tr>
<tr>
<td></td>
<td>115</td>
<td></td>
</tr>
<tr>
<td>Shear modulus, (G_{C,k}) (MPa) (^1)</td>
<td>26</td>
<td>Extrusion direction, wrinkling verification</td>
</tr>
<tr>
<td>Poisson ratio, (\nu_C) (-)</td>
<td>0.30</td>
<td>Assumed value</td>
</tr>
<tr>
<td>Thermal elongation coefficient, (a_{cm}) (K(^{-1})) (^2)</td>
<td>76E-6</td>
<td>(\perp) to extrusion direction</td>
</tr>
</tbody>
</table>

\(^1\) characteristic, \(^2\) average values

Table 3 Core material comparison

<table>
<thead>
<tr>
<th>Property</th>
<th>PUR 145 (Keller et al. 2008)</th>
<th>T92.130 end-grain (Keller et al. 2014)</th>
<th>Balsa end-grain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density, (\rho) (kg/m(^3))</td>
<td>145</td>
<td>135</td>
<td>250</td>
</tr>
<tr>
<td>Tensile strength, (f_{Ct,k}) (MPa) (^1)</td>
<td>1.40</td>
<td>2.11</td>
<td>7.9</td>
</tr>
<tr>
<td></td>
<td>1.40</td>
<td>0.91</td>
<td>0.7</td>
</tr>
<tr>
<td>Shear strength, (\tau_{C,k}) (MPa) (^1)</td>
<td>0.70</td>
<td>1.14</td>
<td>2.3</td>
</tr>
<tr>
<td>Elastic tensile modulus, (E_{Ct,m}) (MPa)</td>
<td>55</td>
<td>175</td>
<td>200</td>
</tr>
<tr>
<td>Shear modulus, (G_{C,k}) (MPa) (^1)</td>
<td>21</td>
<td>30</td>
<td>290</td>
</tr>
<tr>
<td>Thermal elongation coefficient, (a_{cm}) (K(^{-1})) (^2)</td>
<td>50E-6</td>
<td>76E-6</td>
<td>20E-6</td>
</tr>
</tbody>
</table>

\(^1\)\(^-\)\(^-\) = parallel-transverse to extrusion/grain direction
The spatial frame is supported by two elastomer bearings on each side, the support conditions are those of a simple beam. In addition, all supports are secured against uplift since wind uplift forces may be higher than the dead load. The spatial frame was modelled in a finite element software using shell elements; half of the bridge was considered using a symmetry condition, as shown in Figure 6. The geometry was simplified, i.e. the curved edges in the cross section were modelled as sharp corners; detailed verifications demonstrated that this simplification is on the safe side.

**STRUCTURAL VERIFICATIONS**

The structural verifications are based on Swiss standards (SIA), which are in line with Eurocode standards. The ultimate limit (ULS) and serviceability limit (SLS) states were verified using the partial safety factor concept, i.e. applying load factors on the action and resistance factors on the material side. The actions, their combinations and associated load factors were determined according to SIA260 (2013) and SIA261 (2014). Actions taken into account were dead and live loads, snow, wind and temperature. Since resistance factors for FRP materials are defined neither in the Swiss standards nor in the Eurocodes, resistance factors from Eurocomp (Clarke 1996) were adopted, as it is in accordance with the Eurocode design philosophy.

**Resistance Factors**

The resistance factor, $\gamma_R$, according to Eurocomp is composed of three sub-factors, $\gamma_R = \gamma_{M1} \cdot \gamma_{M2} \cdot \gamma_{M3}$, which take the property source, manufacturing method, load duration and operating temperature into account. The sub-factors at ULS were selected as follows: $\gamma_{M1} = 1.15$ (properties derived from tests), $\gamma_{M2} = 1.20$ (vacuum infusion process, fully post-cured), $\gamma_{M3} = 1.20$ or 3.00 for short-term or permanent actions, resulting in $\gamma_R = 1.70$ or 4.10 for short-term or permanent actions. These factors were applied to the characteristic, i.e. 5%-fractile strength and stiffness values of the laminate and core materials in order to obtain the design values. The core and laminate stiffness values were used to calculate the wrinkling resistance of the laminates. For the verification of adhesively-bonded joints, a fourth factor, $\gamma_{M4}$, is applied. The four factors in this case are $\gamma_{M1} = 1.25$ (properties derived from tests), $\gamma_{M2} = 1.50$ (manual application, no adhesive thickness control), $\gamma_{M3} = 1.00$ or 1.50 for short-term or permanent actions, $\gamma_{M4} = 2.00$ (service conditions outside the adhesive test conditions), resulting in $\gamma_R = 3.75$ or 5.60 for short-term or permanent actions.

At SLS, the average stiffness values were reduced by $\gamma_{R} = 1.30$ and creep was considered by a further stiffness reduction factor of 0.41 (Eurocomp, Figure 4.13, CSM/WR worst case at a 50-years' time). Furthermore, a strain limit of 0.2% was taken into account at SLS.

**Ultimate Limit State**

The laminate tensile and compressive resistances were verified, the latter according to the following wrinkling equation (Wiedemann 1996):

$$ f_{w,d} = C_w \cdot \sqrt{E_{L,\|d} \cdot E_{C,\perp,d} \cdot G_{C,\perp,d}} $$

(1)

with $f_{w,d}$ = design value of the wrinkling resistance, $C_w = 0.5$ (Keller et al. 2008), $E_{L,\|d}$ = design value of the laminate elastic modulus, $E_{C,\perp,d}$ = design value of the core elastic modulus, $G_{C,\perp,d}$ = design value of the core shear modulus. Further verifications concerned the core shear stresses and the through-thickness stresses resulting in the curved regions, including laminate-core delamination.

For the stability verification of the compressed frame struts an initial deformation of 1/300 of the buckling length was assumed and a subsequent second order verification was performed.

**Serviceability Limit State**

SLS verifications concerned the deflections, vibrations (eigenfrequencies) and the above mentioned 0.2%-strain limit. The deflections were verified for permanent loads including creep and the live loads. The corresponding deflections limits according to SIA260 (2013) were span/700 and span/600 respectively, in both the longitudinal and transverse directions. Based on the sandwich composition resulting from the ULS verification, all the deflection limits were met with the exception of the deflection of the walkway slab at the largest width. The resulting deflection of span/300 however was assessed as being admissible.
Concerning vibrations, the eigenfrequencies of pedestrian bridges have to exceed 4.5 Hz for vertical and 1.3 Hz for horizontal (transverse) vibrations according to SIA260 (2013). The eigenmodes and eigenfrequencies were calculated taking the dead load and 10% of the live load into account. The resulting first and second eigenmodes are a lateral and a vertical vibration respectively, see Figures 7 and 8, at eigenfrequencies that exceed the limit values.

Figure 7 First eigenmode, lateral oscillation, 3.42 Hz eigenfrequency

Figure 8 Second eigenmode, vertical oscillation, 4.64 Hz eigenfrequency

CONCLUSIONS

The intention of the new TSCB bridge is to create a bridge that meets architectural, structural and manufacturing requirements in an optimized way. The complex shape of the spatial frame can be built up of only one single module which can be manufactured in series. The overhead spatial frame has a large static height that offers a high stiffness and thus allows the laminate and core strengths to be fully used, which in most cases of FRP bridges is not possible due to deflection constraints. The complex, i.e. double-curved shape however also presents some disadvantages. The strength of the expensive carbon fibers cannot be fully used due to the required isotropic lay-up and orthotropic core materials are not ideal since their transverse direction is also stressed. Moreover, the small walkway width at mid-span results in high stresses in this cross section.

Without the architectural intention of exhibiting a black fabric texture and a slight increase of the frame static height and walkway core thickness, the bridge may be conceived at lower cost by using glass fibers. In fact, due to the temperature constraints, significant areas, in particular on the exterior side, need to be painted in a light colour. The selected sandwich construction, however, is an advantageous way to provide the required stiffness to prevent buckling in compressed parts and limit deflections in the walkway slab. Lightweight construction is thus maintained and the use of expensive CFRP materials is minimized. Furthermore, from the architectural point of view, the project demonstrates the liberty in the formal expression and plasticity offered by FRP composite materials.

ACKNOWLEDGEMENTS

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EXPERIMENTAL STUDY ON FLEXURAL PERFORMANCE OF GFRP COMPOSITE SANDWICH BEAMS WITH WOOD-PLASTIC COMPOSITE CORE

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ABSTRACT:
Innovative Fiber Reinforced Polymer (FRP) composite sandwich beams which composed of FRP sheets and Wood-Plastic Composite (WPC) core were manufactured using vacuum infusion molding process. The flexural behaviors of the FRP composite sandwich beams with three different depths ($h = 40$ mm, $85$ mm and $125$ mm) were investigated by four point bending tests. The failure modes obtained from the tests include the crushing of the upper FRP sheet and the tensile failure of the bottom FRP sheet of the FRP composite sandwich beam, and debonding failure between FRP sheets and WPC core was found. The ultimate moment of FRP composite sandwich beams increased by 1.9 times, 14.0 times and 26.9 times compared with those WPC beams, respectively. The results showed that the presence of FRP skins increases the load carrying capacity and result in a more ductile failure behavior.

KEYWORDS:
Fiber Reinforced Polymer (FRP), sandwich beam, Wood-Plastic Composite (WPC), flexural behavior, failure mode.

INTRODUCTION
Wood-Polymer Composite (WPC) materials are becoming popular because of the properties such as resistance against biological deterioration and versatility in processing. They are considered environmentally friendly materials due to the usage of recycled thermoplastics in their compositions and processing residue fillers such as sawdust, rice hulls, or peanut shells. The main part of WPCs is the filler to decrease the cost of the product and increase the stiffness (Clemons 2008). To advance WPCs as a load bearing material, the mechanical properties must be convincingly improved. The published research works investigating the mechanical properties of WPCs were mainly dealing with: compatibilization of wood particles and polymer surfaces (Yang et al. 2007), type and geometry of wood (Bouaffif et al. 2009; Migneault et al. 2008), the effect of processing parameters (Yeh and Gupta 2008; Balasuriya et al. 2001), impact modifier (Oksman and Clemons 1998) and hybridization with short reinforcement fibers (Rizvi and Semeralul 2008; Deng and Tang 2010).

Despite improving the material properties, it is unlikely that WPCs will replace load-bearing members, such as beams or columns, used in building and transportation structures. The limited use is mainly because of the low strength and time-dependent deformation of WPCs compared with traditional structural materials such as steel and concrete. The research conducted by Klyosov (2007) shows that the strength is low because of the poor interfacial bond between wood and polymer. Several studies attempt to improve the compatibility between the wood and polymer interface using various additives. But Lu et al. (2005) shows that using coupling agents alone might not be enough to achieve significant improvement in the mechanical properties of WPCs. The increase in deformation of WPCs over time under sustained load can be attributed to its viscoelastic properties.

One way to increase the mechanical properties of WPCs is to reinforce the surface with some other material with higher mechanical properties such as fiber reinforced polymer (FRP). The study (Dura 2005) conducted quasi-static flexural tests on extruded WPC beams with hollow cross-section reinforced on the top and bottom flanges with one-layer of E-glass FRP. The peak load for the reinforced members increased 2.4 times compared with the unreinforced member, whereas the modulus of elasticity increased by a factor of 1.73. One study conducted by Manalo et al. (2010) involved experimental investigation onto the flexural behavior of glue-laminated fiber
composite sandwich beams with a view of using this material for structural beams. The results shows that the composite sandwich beams in the edgewise position failed with 25% higher bending strength but have 7% lower bending stiffness than beams in the flatwise position. Moreover, Lopez-Anido and Xu (2002) developed a structural system based on the concept of sandwich construction with strong and stiff FRP composite skins bonded to an inner glulam panel. These examples show that the concept of composite sandwich panels to form a structural beam is highly practical.

This paper presents the results of an experimental investigation on the behavior of FRP composite sandwich beams with WPC core. The composite sandwich beams are fabricated by vacuum infusion molding process (VIMP) and subject to static 4-point bending test to determine their flexural properties. The load–deflection behavior, strength and failure mechanisms of the FRP composite sandwich beams are reported. The effects of the number of core layers on the strength and stiffness of FRP composite sandwich beams are also discussed.

EXPERIMENTAL PROGRAM
Material and specimens

Alkali-free glass fiber, wood-plastic composite, and unsaturated resin were selected to manufacture the composite sandwich beams with different sections. Figure 1 shows the manufacturing of FRP composite sandwich beam specimens. Each WPC core was wrapped with one layer of a bidirectional fabric (0/90°) with 800 g/m² of areal density, which was called sandwich panel element. Two or three sandwich panel elements were assembled together, and then the glass bidirectional fabric was covered on it by vacuum infusion molding process to manufacture a complete GFRP composite sandwich beam. The descriptions of the test specimens are listed in Table 1.

Table 1 Specimens, the ultimate bending strength and ultimate moment

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Illustration</th>
<th>Number of specimen</th>
<th>D (mm)</th>
<th>B (mm)</th>
<th>Length (mm)</th>
<th>Support span (mm)</th>
<th>$P_u$ (kN)</th>
<th>$M_u$ (kN·m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WPC</td>
<td></td>
<td>2</td>
<td>40</td>
<td>81</td>
<td>1400</td>
<td>1200</td>
<td>1.8</td>
<td>0.4</td>
</tr>
<tr>
<td>1W-FRP</td>
<td></td>
<td>2</td>
<td>41</td>
<td>82</td>
<td>1400</td>
<td>1200</td>
<td>3.5</td>
<td>0.78</td>
</tr>
<tr>
<td>2W-FRP</td>
<td></td>
<td>2</td>
<td>85</td>
<td>85</td>
<td>1400</td>
<td>1200</td>
<td>25.2</td>
<td>5.67</td>
</tr>
<tr>
<td>3W-FRP</td>
<td></td>
<td>2</td>
<td>125</td>
<td>85</td>
<td>1400</td>
<td>1200</td>
<td>48.5</td>
<td>10.9</td>
</tr>
</tbody>
</table>
Test set-up, measurements and procedure

The 4-point static bending test on composite sandwich beams was performed in accordance with the ASTM C393-00 standard (2000). The distance between two loading points was 300mm and the length of the shear span was 450mm. A loading speed of 2 mm/min was employed. The actual test set-up and instrumentation for the static flexural test of composite sandwiches are shown in Figure 2. Before each test, the loading pins were set to almost touch the top surface of the composite sandwich specimen. The applied load, displacement and strains were recorded. The failure behaviors were also monitored and recorded.

EXPERIMENTAL RESULTS

Failure behavior

Experimental investigation showed that the GFRP composite sandwich beams exhibited some failure behaviors. These failure modes of the composite sandwich beams are shown in Figure 3.

Figure 2 Test set-up and measurement of flexural test

Figure 3 Failure of composite sandwich beams.

Specimen WPC failed in tension on the bottom, leaving an irregular fracture surface in the middle region of the specimen. The cracks of the compressive GFRP sheets were observed in specimen 1W-FRP. These cracks were also observed on the upper GFRP sheets for specimen 2W-FRP and 3W-FRP. When these cracks appeared, a significant drop in the load was observed. These cracks originated at the upper GFRP sheets and progressed with the application of load, but these cracks did not cause immediate failure. As these cracks developed, the damage
on the specimen increased, thereby, increasing the deflection. The specimen continued to carry load even after compressive failure of the upper GFRP sheets as shown by the cracks which developed near the loading point. The continuous application of load caused the fracture of the upper GFRP sheets. Splitting of the tensile GFRP sheets were also then observed. Final failure of the GFRP sandwich beams was due to tensile failure of the GFRP sheet on the bottommost. And the results of the experimental investigation with 3W-FRP also showed slipping occurred on the WPCs. The results show that the GFRP composite sandwich beams should improve the interfacial bonding properties.

**Load–deflection curve**

The load and mid-span deflection behavior of all specimens under 4-point static bending is shown in Figure 4. The figure shows that the deflection of specimen WPC increased almost linearly with load up to final failure, with an applied load of 1.8kN and a mid-span deflection of 43.9 mm. The figure also shows that the load of specimen 1W-FRP increased linearly with deflection before the load reached 2.8kN. The first crack occurred in specimen 1W-FRP at an applied load of 2.7kN, and the beam was still able to carry load until final failure.

The load of 2W-FRP increased linearly with deflection before 18.2kN. Then the load dropped because the slipping of WPC cores was occurred. After the cracks occurred on the upper GFRP skins, the beams were still able to carry load but showed large deflection until final failure of the bottom. A similar load deflection behavior was observed in specimen 3W-FRP. The load deflection curves for specimen 3W-FRP are almost linear until the load of 35.2kN and 41kN. As the loading continues, the Specimen 3W-FRP continued to carry load even after compressive failure of the upper GFRP sheets. The specimen behaved non-linearly with a reduced stiffness up to final failure.

![Figure 4 Load and mid-span deflection relationship of specimens](image)

**CONCLUSIONS**

The flexural behavior of GFRP composite sandwich beams with WPC cores was determined through experimental investigation. The ultimate load of specimens W-FRP, 2W-FRP and 3W-FRP increased by 1.9 times, 14 times and 26.9 times compared with specimen WPC, respectively. The results show that GFRP composite sandwich beams with WPC core resulted in a stronger and more stable section than individual WPCs. The results also suggest that the presence of GFRP skins increases the load carrying capacity and result in a more ductile failure behavior. The final failure of the specimen is due to tensile failure of the GFRP sheets. Finally, the results of this study demonstrated the high possibility of developing a structural beam from GFRP composite sandwich structures. Currently, research is being conducted to make full use of the GFRP composite sandwich beams with the objective of exploring the practical application of this for composite structures.

**REFERENCES**


CYCLIC LOADING RESPONSE OF PRECAST CONCRETE SANDWICH SHEAR WALLS WITH FRP CONNECTORS

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ABSTRACT
This paper investigated cyclic loading responses of three full-scale shear walls, including two precast concrete sandwich shear walls (PCSSWs) with FRP connectors and one cast-in-place (CIP) control shear wall. Horizontal connections of the PCSSW specimens were achieved through one or two rows of connecting bars spliced by steel sleeves. Test results revealed that all specimens failed in bending and exhibited stable hysteresis loops and similar energy dissipation. The difference in load-carrying capacity between the three specimens was less than 2%. Besides, all specimens behaved in a ductile manner, the ductility of the two PCSSWs (2.4 and 2.65) was slightly higher than that of the CIP control specimen (2.35). No damage was observed in the FRP connectors during the whole test. Moreover, a restoring force model was developed based on the test results. Overall, the PCSSW specimens showed better seismic performance, especially the specimen with single-row connecting reinforcements.

KEYWORDS
Precast concrete, sandwich shear walls, FRP connectors, row number of connecting bars, cyclic test, load-carrying capacity, ductility, restoring force model.

INTRODUCTION
As one of the most appropriate structures for residential buildings, precast concrete can be divided into precast concrete solid shear walls and precast concrete sandwich shear walls (PCSSWs). Among these, PCSSWs have been in use for several decades, first appearing in North America more than 50 years ago (K. Seeber et al. 1997). A typical PCSSW consists of external and internal precast concrete panel, called wythes, separated by an insulation layer (PCI 2011). The concrete wythes are connected through the insulation layer by connectors. Based on good mechanical properties and thermal insulation, fibre-reinforced polymer (FRP) connector is a common choose in PCSSWs. At present, PCSSWs with FRP connectors are commonly used in commercial and institutional structures as well as residential construction (Holmes et al. 2005) since they have excellent durability and fire-resistance on the premise of adequate thickness of cover, and can achieve a same life span of thermal insulation and load-bearing.

Although PCSSWs have been applied in some seismically active regions, investigations on the behaviour of them are very scarce. Therefore, it is necessary to study the mechanical properties, including seismic behaviour. Existing researches of precast concrete shear walls mainly concentrated on solid shear walls. Soudki et al. (1995) tested six full-scale specimens to investigate the behaviour of mild steel connections for precast concrete shear wall panels. The horizontal connection was reinforced by two continuity reinforcing bars spliced by sleeve, welded to steel angle or bolted to tube section. Results from the test revealed that precast shear walls all behaved satisfactorily. Qian et al. (2010) conducted quasi-static tests of five concrete shear wall specimen with different methods of vertical reinforcement splicing. According to the analysis, splicing sleeve transferred stresses of vertical reinforcement effectively.

There are some general provisions about designing precast concrete shear walls but no detailed provisions for the PCSSWs investigated in this study in ACI 318, EC 8, NZS 3101, and the Chinese code for seismic design of buildings.

Since 2007, our research team developed a kind of FRP connector and finished a series of tests to investigate its mechanical properties, durability and fire-resistance. Reliable performance of the connector was confirmed through above-mentioned tests. On this basis, this paper conducted cyclic loading tests under high axial load ratio.
(0.4) to investigate the cyclic loading response of PCSSWs with the new FRP connectors. In addition, the influence of row number of connecting bars was also discussed.

**EXPERIMENTAL PROGRAM**

**Specimen Details and Materials**

A total of three specimens were tested under cyclic loading. These three specimens comprised two PCSSWs of size 2000m x 2900m x 310mm (Specimen ID: TW1, TW2) and one CIP shear wall of size 2000m x 2900m x 200mm (Specimen ID: SW1). The PCSSW was composed of a 50-mm-thick extruded polystyrene (XPS) foam insulation layer with FRP connectors encased in a 200-mm-thick internal wythe and a 60-mm-thick external wythe. A precast specimen consisted of upper wall and ground beam joined by a horizontal connection. The horizontal connection of specimen TW1 was reinforced by two rows of continuity reinforcing bars while specimen TW2 only had one single row of connecting reinforcements. The dimensions and reinforcement details are shown in Fig. 1. A 20-mm-thick gap was reserved between the external wythe and the top beam. The concrete mixture was designed for a cube compressive strength of 35 MPa for all specimens. As for reinforcing bars, the yield strength was around 400MPa and the ultimate strength was about 540MPa.

**Test setup and test procedure**

The specimens were fixed to the rigid floor by anchor bolts as shown in Fig 2 and were subjected to a combination of horizontal and vertical loading. A constant axial load was applied to the specimens by using a vertical 10000kN capacity hydraulic actuator. The level of load on the wall was 40% of its capacity, which was calculated by multiplying the gross section of concrete area (not the XPS core) by the average compressive strength of all tested specimens.
panels. The lateral cyclic loading was divided into two phases. Initially, loading was applied under load control with increments of 50 or 100 kN. Following the cracking of concrete, the behaviour of the specimen became inelastic and loading was switched to displacement control. The second phase consisted of successive cycles progressively increasing 0.25% story drift. Three fully reversed cycles were applied at each story drift level. The test was continued under displacement control until failure of the specimen occurred or lateral load was lower than 85% of the capacity.

![Figure 2 Reversed displacement control cyclic history](image)

**EXPERIMENTAL RESULTS**

**Failure Pattern**

All shear walls failed in bending. At the time of failure, yielding of longitudinal reinforcement was followed by concrete crushing and spalling at the bottom of wall. (Fig. 3a and b) No pull-out of the continuity reinforcement from the sleeve was found in the precast specimens TW1 and TW2. The damage of both precast specimens was concentrated on internal wythes. Little damage of external wythes or connectors were observed during the whole test, which indicated the low participation level of external wythes in resisting lateral load. (Fig. 3c)

![Fig 3 Typical failure patterns](image)

**Hysteresis characteristics**

The lateral load-displacement hysteresis curves of all specimens are given in Fig. 4. The hysteresis hoops of PCSSW specimens were quite similar to those of the control specimen, which was considered a satisfactory performance. For all specimens, the hysteresis hoops exhibited pinching to some extent. Area of hysteresis loops of specimen TW2 was slightly larger than that of specimen TW1.

![Figure 4 Hysteresis curves](image)
The yield load, peak load, and corresponding displacement of three specimens were very close, which can be seen in skeleton curves (Fig. 5). The bearing capacities were 1455kN, 1473kN, 1417kN for specimen TW1, TW2 and SW1, respectively. The small difference between precast specimens and monolithic specimen lied in the presence of splice sleeve. All specimens exhibited similar trend of stiff degradation.

![Skeleton curves](image)

**Figure 5 Skeleton curves**

### Displacement ductility and deformability

The displacement ductility is the ratio of the displacement at failure to the yield displacement. Table 1 listed the yield displacement $\Delta_y$, the displacement at failure $\Delta_u$ and corresponding displacement ductility $\mu = \Delta_u / \Delta_y$. The yield displacement was determined according to the criteria suggested by Park (1989). The failure of the specimen was defined as the point at which the applied lateral load was 85% of the peak load. According to the results in Table 2, the following conclusions could be drawn:

1. The displacement ductility factor of monolithic specimen was 2.35, which is lower than the displacement ductility factor of precast specimens. This difference revealed that splice sleeves confined the concrete near the bottom of wall and delayed the crushing, thus increased the displacement ductility of specimen.

2. The displacement ductility factor of specimen T1 was 2.40; for specimen T2, it was 2.65. PCSSW with single-row connecting reinforcements exhibited better displacement ductility than that with double-row connecting reinforcements.

3. The ultimate deformability of PCSSWs was slightly larger than the monolithic shear wall.

![Table 1](image)

**Table 1 Displacement ductility values of test specimens**

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>$\Delta_y$ (mm)</th>
<th>$\Delta_u$ (mm)</th>
<th>$\mu = \Delta_u / \Delta_y$</th>
<th>$\mu_{ave}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>SW1</td>
<td>11.4</td>
<td>29.1</td>
<td>2.6</td>
<td>2.35</td>
</tr>
<tr>
<td>TW1</td>
<td>14.1</td>
<td>29.3</td>
<td>2.1</td>
<td>2.40</td>
</tr>
<tr>
<td>TW2</td>
<td>14.1</td>
<td>34.5</td>
<td>2.6</td>
<td>2.65</td>
</tr>
</tbody>
</table>

### Restoring force model

By analysis of hysteresis hoops, skeleton curves and characteristic loads of two PCSSW specimens, a four-linear restoring force model was proposed, as shown in Fig. 6. The normalized characteristic parameters for the P-$\Delta$ hysteretic model were listed in Table 2. In this table, $P_{cr}$, $P_y$, $P_{max}$ and $P_u$ symbolized cracking load, yielding load, peak load and failure load, respectively; $\Delta_{cr}$, $\Delta_y$, $\Delta_{max}$ and $\Delta_u$ were lateral displacement corresponding to these loads, respectively. In Fig. 6, “+” and “-” in front of letters denoted the characteristic values in positive and negative directions, respectively.

![Table 2](image)

**Table 2 Displacement ductility values of test specimens**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$P_{max}$</th>
<th>$P_{cr}$</th>
<th>$P_y$</th>
<th>$P_u$</th>
<th>$\Delta_{max}$</th>
<th>$\Delta_y$</th>
<th>$\Delta_u$</th>
<th>$\Delta_p$</th>
<th>$2\Delta$</th>
<th>$3\Delta$</th>
<th>$4\Delta$</th>
<th>$5\Delta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>TW1</td>
<td>+ 1.00</td>
<td>0.61</td>
<td>0.78</td>
<td>0.99</td>
<td>0.41</td>
<td>1.00</td>
<td>0.28</td>
<td>0.53</td>
<td>1.36</td>
<td>0.12</td>
<td>1.69</td>
<td>1.22</td>
</tr>
<tr>
<td></td>
<td>- 1.00</td>
<td>0.69</td>
<td>0.72</td>
<td>1.00</td>
<td>0.23</td>
<td>1.00</td>
<td>0.28</td>
<td>0.45</td>
<td>1.0</td>
<td>0.07</td>
<td>6.92</td>
<td>1.06</td>
</tr>
<tr>
<td>TW2</td>
<td>+ 1.00</td>
<td>0.56</td>
<td>0.80</td>
<td>0.93</td>
<td>0.36</td>
<td>1.00</td>
<td>0.25</td>
<td>0.57</td>
<td>1.56</td>
<td>0.15</td>
<td>2.66</td>
<td>1.38</td>
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<tr>
<td></td>
<td>- 1.00</td>
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<td>0.67</td>
<td>1.00</td>
<td>0.43</td>
<td>1.00</td>
<td>0.30</td>
<td>0.38</td>
<td>1.00</td>
<td>0.10</td>
<td>2.03</td>
<td>0.71</td>
</tr>
</tbody>
</table>
Main hysteretic rules were expressed as follows:

(1) The skeleton curves of specimens were simplified into four-fold line in both positive direction and negative direction. The characteristic points were cracking point, yielding point, peak point and failure point.

(2) Before cracking, the initial stiffness $K_1 (= P_{cr}/\Delta_{cr})$ was taken as the loading stiffness, and stiffness degradation and residual deformation were not taken into account during unloading. Reloading rules in the negative direction were that the curves directly pointed to a cracking point in the negative direction.

(3) During the stage between cracking point and yield point, $K_2 (= (P_y - P_{cr})/(\Delta_y - \Delta_{cr}))$ was taken as loading stiffness, and stiffness degradation and residual deformation were considered.

(4) Post-yielding stiffness $K_3 (= (P_{max} - P_y)/(\Delta_{max} - \Delta_y))$ was defined as loading stiffness after yield point. Loading stiffness $K_4 (= (P_u - P_{max})/(\Delta_u - \Delta_{max}))$ became negative stiffness after peak point.

(5) The initial stiffness $K_1$ was taken as unloading stiffness by reduction factor $\beta$. Here, $\beta = (\Delta_y/\Delta_m)v$, where $\Delta_m$ was the maximum displacement experienced, and $v$ was regressed from the test results.

(6) Reloading rules in the negative (positive) direction after post-cracking unloading were that the curves directly pointed to pivot pinching point, $M (N)$ (the ordinates of which are listed in Table 4) in the positive (negative) direction, then pointed to the maximum previous displacement point, and then took $K_2, K_3$ as the loading stiffness pointing to the skeleton curve.

![Figure 6 Restoring force model](image)

Energy dissipation capacity

The energy dissipation capacity of a shear wall is a function of the area under the load-displacement curve, and indicates the degree of effectiveness of the wall to withstand earthquake loading. The cumulative energy dissipated during the test was plotted in Fig. 7. By comparison of cumulative energy dissipated after same number of cycles, it can be found that the energy dissipation capacity of the PCSSW with single-row connecting reinforcements (70.2 kN-m) was higher than that of PCSSW with double-row connecting reinforcements (44.7 kN-m). This difference may lie on the connecting reinforcements at the connection area. To be more specific, the double rows of longitudinal reinforcements were replaced by one single row of connecting reinforcements in specimen T2. Consequently more reinforcements existed in the area of anchorage length.

![Figure 7 Energy dissipation](image)

CONCLUSIONS

Based on test results presented here and from the observations during the test, the following conclusions may be drawn.
(1) All specimens failed in bending. The connecting reinforcements didn’t rupture or pull-out from the sleeve. The external wythe participate in resisting lateral load to a very small degree and had little damage. In addition, no damage was observed in the FRP connectors during the whole test.

(2) The difference in load-carrying capacity between three specimens was less than 2%.

(3) Each specimens exhibited stable hysteresis behavior until the onset of failure, followed by pinched loops during post-failure cycles.

(4) The displacement ductility of three specimens was larger than 2, which indicated a ductile manner.

(5) Based on the above, precast concrete sandwich shear walls have similar seismic performance to the CIP shear wall. It is feasible to use splice sleeves as connections in precast walls, and simply the connection into single row.

ACKNOWLEDGMENTS

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REFERENCES


BOND BEHAVIOUR OF COMPOSITE SANDWICH PANEL AND EPOXY POLYMER MATRIX

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ABSTRACT
Fibre composite sandwich panel made up with glass fibre reinforced polymer skins and phenolic foam core can be glued or cast together to produce a large structural beam section. To ensure the structural integrity and composite action, the sandwich panels should be effectively bonded with polymer matrix. However, the bond behaviour between sandwich panel and polymer matrix is not well understood. This paper experimentally investigated the effect of epoxy polymer matrix properties, bond length, bond thickness, and bond width on the bond behaviour, and evaluated the optimal parameters for effective bonding. The experimental program was designed by Taguchi method to reduce the number of experiments. Results showed that the polymer matrix consist with 40% filler and 60% resin (by volume) is the optimal binder. A bond length of at least 70 mm and bond thickness of 5 mm were found effectively to utilise the strength of the composite sandwich panel. The bond width however has insignificant effect on the bond strength.

KEYWORDS
Bond behaviour, sandwich panel, epoxy, filler, effect of design parameters.

INTRODUCTION
In recent years, a composite sandwich structure has become an effective alternative structural element for industrial applications due to its excellent design flexibility and good balance between rigidity and lightness (Manalo 2013). Applying this type of composite sandwich system to develop cost-effective civil infrastructure including bridge beams, railway sleepers and floating waler is now being explored (Ferdous et al. 2015). As sandwich panels are generally produced with limited thicknesses for production efficiency, a number of panels are bonded together with an epoxy polymer matrix to achieve the desired cross-section. This combination can be used for manufacturing bridge girders, railway sleepers and similar structures. Despite the cost benefits of vinyl-ester and polyester compared with epoxy, they are not suitable if excellent mechanical and thermal properties, superior resistance to humidity, low shrinkage and high elongation are required to produce a durable and flexible polymer matrix (Loos 2014). This study used epoxy resin for the polymer matrix because of its superior mechanical, durability and thermal properties and examined whether such a matrix is suitable for binding composite sandwich panels. In this process, the structural integrity and composite action of both the panels and matrix used are important in terms of transferring the load from one panel to another through the bond as de-bonding between a sandwich panel and binder interface can lead to premature failure of the structures.

Studies conducted on using externally bonded FRP/steel plates to rehabilitate existing structures (Chen and Teng 2001; Yuan et al. 2004) have found that the properties of the adhesive materials, and the bond length, thickness and width are some of the parameters that influence bond behaviour. However, those studies were conducted for the specific purpose of retrofitting existing structures where the FRP was bonded with concrete by a very thin layer of resin (< 1 mm) (Arenas et al. 2010) which was applied on the surface of concrete before placing FRP sheets. In the present study of a sandwich panel bonded with an epoxy polymer matrix, the properties and thickness of the matrix and casting method were different, which could affect the bond performance, the findings from retrofitting studies may not be applicable. Therefore, the main motivation for this study was to determine how the matrix properties, bond length, thickness and width affect the bond between a composite sandwich panel and epoxy polymer matrix.
MATERIALS AND METHODS

Materials

Sandwich panel

The composite sandwich panel consists of GFRP composite layers (skins) bonded to a phenolic core. The top and bottom skins were 2 mm thick with a fibre volume ratio of 45%. The phenolic foam core material came from natural plant (non-food) products derived from vegetable oils and plant extracts and was chemically bonded with the polymer resin (Van Erp and Rogers 2008). The properties of the constituent materials of sandwich panel are determined by (Manalo et al. 2012) and the necessary properties are provided in Table 1.

Table 1: Properties of the constituents of sandwich panel

<table>
<thead>
<tr>
<th>Type of test</th>
<th>Properties</th>
<th>GFRP skin</th>
<th>Phenolic core</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Longitudinal</td>
<td>Transverse</td>
</tr>
<tr>
<td>Tensile</td>
<td>Elastic modulus (GPa)</td>
<td>15.38</td>
<td>12.63</td>
</tr>
<tr>
<td></td>
<td>Peak stress (MPa)</td>
<td>246.80</td>
<td>216.27</td>
</tr>
<tr>
<td>Compressive</td>
<td>Elastic modulus (GPa)</td>
<td>16.10</td>
<td>9.95</td>
</tr>
<tr>
<td></td>
<td>Peak stress (MPa)</td>
<td>201.75</td>
<td>124.23</td>
</tr>
</tbody>
</table>

Epoxy polymer matrix

Polymer matrix was prepared by mixing resin and filler. Fillers were added from 30% to 50% (by volume) with the resin in increments of 10% as, from a previous study by the authors (Ferdous et al. 2016), mixes in that range were found to be the most suitable. The two main components of these resin systems were a DGEBA type epoxy resin and amine-based curing agent, with three different filler materials a fire retardant filler, hollow microsphere and fly ash, mixed together in approximate percentages to obtain an effective filler mix. The properties of the three mixes are provided in Table 2.

Table 2: Properties of polymer matrices

<table>
<thead>
<tr>
<th>Properties</th>
<th>Properties of different polymer matrices</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexural strength (MPa)</td>
<td>30% Filler 40% Filler 50% Filler</td>
</tr>
<tr>
<td>% Failure strain in flexure</td>
<td>1.9 1.2 1.0</td>
</tr>
<tr>
<td>Density (gm/cm³)</td>
<td>1.34 1.38 1.43</td>
</tr>
<tr>
<td>% Porosity</td>
<td>0.06 0.62 0.98</td>
</tr>
</tbody>
</table>

Taguchi Design of Experiments

The experimental program is designed in accordance with Taguchi method which is an effective tool for the design of experiments. Among the different parameters influencing bond strength such as matrix properties, bond length, bond thickness, bond width, surface roughness, and curing temperature, method and time, this study primarily focuses on the matrix properties and influence of bond dimensions on bond behaviour. Therefore, the design parameters considered in this study are the matrix properties, and bond length, thickness and width, each with three levels are shown in Table 3. These three levels of each parameter are considered for understanding the influence of the variation of matrix properties and bond dimensions.

Table 3: Selected design parameters and their levels

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Levels</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>A % Filler in epoxy polymer matrix (by volume)</td>
<td>30</td>
</tr>
<tr>
<td>B Bond length (mm)</td>
<td>40</td>
</tr>
<tr>
<td>C Bond thickness (mm)</td>
<td>3</td>
</tr>
<tr>
<td>D Bond width (mm)</td>
<td>20</td>
</tr>
</tbody>
</table>

As four independent design parameters, each with three levels that have no interactions with each other require a minimum of 9 experiments instead of 81 (i.e. 3⁴) as used in the traditional factorial experimental design, an $L_9(3^4)$ orthogonal array should be the best choice. The 9 experiments selected based on this array are shown in Table 4 (Olivia and Nikraz 2012).
Table 4: orthogonal array used for experimental design

<table>
<thead>
<tr>
<th>Experiment No.</th>
<th>% Filler</th>
<th>Bond length (mm)</th>
<th>Bond thickness (mm)</th>
<th>Bond width (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>30</td>
<td>40</td>
<td>3</td>
<td>20</td>
</tr>
<tr>
<td>2</td>
<td>30</td>
<td>70</td>
<td>5</td>
<td>40</td>
</tr>
<tr>
<td>3</td>
<td>30</td>
<td>100</td>
<td>10</td>
<td>80</td>
</tr>
<tr>
<td>4</td>
<td>40</td>
<td>40</td>
<td>5</td>
<td>80</td>
</tr>
<tr>
<td>5</td>
<td>40</td>
<td>70</td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td>6</td>
<td>40</td>
<td>100</td>
<td>3</td>
<td>40</td>
</tr>
<tr>
<td>7</td>
<td>50</td>
<td>40</td>
<td>10</td>
<td>40</td>
</tr>
<tr>
<td>8</td>
<td>50</td>
<td>70</td>
<td>3</td>
<td>80</td>
</tr>
<tr>
<td>9</td>
<td>50</td>
<td>100</td>
<td>5</td>
<td>20</td>
</tr>
</tbody>
</table>

The influence of the design parameters on the final output depends on the signal-to-noise ratio (SNR) that can be calculated by Eq. (1).

\[ SNR = -10\log\left(\frac{1}{n} \sum_{j=1}^{n} \frac{1}{y_j^2}\right) \]  

where, ‘j’ the trial number, ‘n’ is the total number of trials and ‘y_j’ is the magnitude of jth trial.

Sample Preparation and Test Setup

In order to investigate the effects of different parameters on bond behaviour, a special specimen configuration was designed to perform bond tests (Figure 1a). MTS Insight 100 kN capacity twin-column testing machine was used that can reliably perform standard tests such as peel, tear, shear, tensile, compression, and flexure. Five specimens in each category (Figure 1b) were tested in order to obtain reliable results. The bond between a sandwich panel and polymer matrix was investigated by a direct compression test in accordance with the most relevant test standard ASTM D905 at a speed of 1.2 mm/min. Under compression loading (Figure 1c), the sandwich panels transferred stresses to the matrix from two opposite directions that generated a shear stress in the polymer matrix.

RESULTS

Failure Behaviour

Three types of failure observed for the specimens (Figure 2) under compression loading were: (a) adhesion failure, where the bond failed at the interface between the polymer matrix and sandwich panel; (b) cohesion failure, where the polymer matrix fractured; and (c) panel failure, where the sandwich panel failed. An adhesion failure was observed for the samples in Expt. - 1, 6 and 8 (Figures 2(a), (f) and (h)) whereas the specimens were failed due to the cohesion of polymer matrix in Expt. - 3, 5 and 7 (Figures, 2(c), (e) and (g)). The third mode of failure was the failure of sandwich panel which was observed in the samples for the Expt. - 2, 4 and 9 (Figures, 2(b), (d) and (i)). Adhesion failures occurred in Expt. - 1, 6 and 8 which indicated that a weak chemical bond formed at the interface, a kind of failure not desirable when a good bond is expected. On the other hand, the cohesion failures observed in Expt. - 3, 5 and 7 indicated strong chemical bond at the interface of a sandwich panel and polymer matrix. However, this type of failure does not ensure effective use of the full strength capacity of sandwich panel which is the main structural component. Therefore, the effective utilisation of sandwich panel in the structural application deserve a failure in it which was observed in the Expt. - 2, 4 and 9. The skin delamination followed by core tension failure of a sandwich panel not only ensures utilisation of the full capacity of the panel but also represents a very strong bond between panel and the matrix.
Analysis of the Results

Under compression loading the panels were subjected to high shear stress in the joint area and the bond strength between panel and matrix can be evaluated by Eq. (2).

$$ Bond \text{ strength} = \frac{Failure \text{ load}}{Bond \text{ length} \times Bond \text{ width}} $$  \hspace{1cm} (2)

The failure loads and strengths obtained from the nine sets of experiments, their standard deviations, SNRs and overall mean of SNRs are presented in Table 5.

Table 5: Average failure loads, strengths, standard deviations (SDs) and SNRs of experiments

<table>
<thead>
<tr>
<th>Expt. no.</th>
<th>Average failure (kN)</th>
<th>Failure strength (MPa)</th>
<th>SNR</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Loads    SD  Mode</td>
<td>T-1</td>
<td>T-2</td>
</tr>
<tr>
<td>1</td>
<td>8.18     1.83 Adhesion</td>
<td>4.41</td>
<td>5.54</td>
</tr>
<tr>
<td>2</td>
<td>35.51    1.96 Panel</td>
<td>6.42</td>
<td>6.64</td>
</tr>
<tr>
<td>3</td>
<td>81.52    2.68 Cohesion</td>
<td>5.08</td>
<td>5.02</td>
</tr>
<tr>
<td>4</td>
<td>31.30    0.87 Panel</td>
<td>4.99</td>
<td>4.87</td>
</tr>
<tr>
<td>5</td>
<td>14.11    1.34 Cohesion</td>
<td>5.40</td>
<td>4.37</td>
</tr>
<tr>
<td>6</td>
<td>37.43    2.01 Adhesion</td>
<td>5.04</td>
<td>4.69</td>
</tr>
<tr>
<td>7</td>
<td>13.45    0.91 Cohesion</td>
<td>4.54</td>
<td>4.17</td>
</tr>
<tr>
<td>8</td>
<td>60.16    23.74 Adhesion</td>
<td>6.31</td>
<td>7.16</td>
</tr>
<tr>
<td>9</td>
<td>22.07    0.84 Panel</td>
<td>5.73</td>
<td>5.47</td>
</tr>
</tbody>
</table>

Overall mean of signal-to-noise ratio (SNR) = 13.84

Note: T-1 to T-5 indicate Test-1 to Test-5 of replicate samples

Although Expt. - 2 showed superior performance to the others in terms of bond strength, this did not indicate the optimum set of parameters. Using the SNR values presented in Table 5, it was possible to determine the effect of individual parameters at different levels. The SNR of a particular design parameter at a specified level was calculated and termed the level mean SNR, are given in Table 6.
Table 6: Level mean SNR of each design parameter at different levels

<table>
<thead>
<tr>
<th>Level</th>
<th>Level mean SNRs</th>
<th>SNR (% Filler)</th>
<th>SNR (Bond length)</th>
<th>SNR (Bond thickness)</th>
<th>SNR (Bond width)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>14.57</td>
<td>13.26</td>
<td>13.14</td>
<td>14.11</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>13.70</td>
<td>14.14</td>
<td>14.87</td>
<td>13.94</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>13.24</td>
<td>14.11</td>
<td>13.50</td>
<td>13.46</td>
<td></td>
</tr>
<tr>
<td>∆SNR</td>
<td>1.33</td>
<td>0.88</td>
<td>1.73</td>
<td>0.65</td>
<td></td>
</tr>
<tr>
<td>Rank</td>
<td>2</td>
<td>3</td>
<td>1</td>
<td>4</td>
<td></td>
</tr>
</tbody>
</table>

The difference between the maximum and minimum level mean SNR (ΔSNR) and ranks, which indicated the order of the influence of the four design parameters on bond strength, were calculated (Table 6). The highest ΔSNR value, which was assigned rank 1, indicated that the bond thickness had the greatest influence on bond strength, with the properties of the polymer matrix and bond length ranked second and third, respectively, followed by the bond width.

DISCUSSION

The variation of level mean SNR with respect to the each level of design parameter are plotted in Figure 3.

![Figure 3: Effect of design parameters on bond strength](image)

**Effect of Matrix Properties**

It can be observed in Figure 3(a) that increasing the filler content from 30% to 50% gradually decreased the SNR response which meant that an increase in the amount of filler gradually weakened the bond between a sandwich panel and polymer matrix. The adhesive properties of the polymer matrix depended primarily on the amount of resin which gradually decreased with increases in the filler content, similar to the tensile or flexural properties of the polymer matrices provided in Table 2. The flow-ability of the matrix had an effect on the quality of the bond; For example, Expt. - 8 produced the most inconsistent results (a large standard deviation in Table 5) due to the formation of voids in the joint during casting because the matrix was composed of 50% filler that had less workability and promoted to create voids while flowing. As the other two matrices (with 30% and 40% fillers) produced quite consistent results, they were strong matrices for use in the final application. However, as the economical design of a polymer matrix suggests an optimal use of resin, a matrix composed of 40% filler and 60% resin was the preferred choice.

**Effect of Bond Length**

Compression of the sandwich panel was transferred to the polymer matrix through shear stresses along the bond length. Results showing an increase in failure strength when the bond length increased from 40 mm to 70 mm while an insignificant variation was observed between 70 mm and 100 mm (Figure 3b). This led to an important aspect of effective bond length beyond which an extension of the bond length cannot increase the ultimate bond strength. From the present study, it can be concluded that the experimentally evaluated effective bond length for a sandwich panel and polymer matrix was approximately 70 mm.

**Effect of Bond Thickness**

Increasing the bond thickness from 3 mm to 5 mm increased the bond strength but, from 5 mm to 10 mm, decreased it, as is evident in Figure 3(c). The lower bond strength at 3 mm than 5 mm was due to the formation of voids during casting, caused by the polymer matrix not being able to flow smoothly through the thinner bond line. A close inspection of the failure modes of the specimens presented in Figure 2 and Table 5 indicates that the bond thickness had a very strong influence on failure behaviour. An observation of the failure modes of thicker bond
specimens (Expt. - 3, 5 and 7) indicates that the failure path of the polymer matrix was not vertical but rather angular. A decrease in failure strength due to a thicker bond could be attributed to the greater eccentricity of the load path. Similarly, thinner bond specimens (Expt. - 1, 6 and 8) exhibited less eccentricity of the load path, the primary cause of adhesion bond failure. The failure of a sandwich panel with a 5 mm bond thickness represented the desired failure mode and a strong bond between the two components.

**Effect of Bond Width**

The effect of the bond width on failure strength was investigated using three different widths of 20 mm, 40 mm and 80 mm. The responses from the analysis showed that an increase in the bond width slightly decreased the ultimate failure strength, i.e., the utilisation efficiency of the combined action of a sandwich panel and polymer matrix marginally reduced. However, variations among the responses of the three different widths were insignificant (Figure 3d) compared with those of the other variables in this study. Consequently, an increase in bond width could increase the load-carrying structure but its effect on the ultimate failure strength was minimal which indicated that a wider sandwich structure bonded with polymer matrix could be designed for a unit width.

**CONCLUSIONS**

From the bond behaviour of sandwich panel and epoxy polymer matrix, the following conclusions are drawn-

1. The bond strength between a sandwich panel and epoxy polymer matrix decreases with an increase in the amount of filler due to the reduced adhesive properties of the matrix. To minimise voids and make the matrix economical, a polymer matrix with 40% filler and 60% resin is preferable.

2. The bond strength is improved by increasing the bond length up to 70 mm, beyond which the extension of the bond length cannot increase the bond strength.

3. The thickness of the polymer matrix has the greatest influence on the bond performance and failure mode. The failure of a sandwich panel with a 5 mm bond thickness indicates a very strong bond between the panel and matrix, which is the desired mode of failure for the effective utilisation of panels.

4. The bond width has an insignificant influence on the bond behaviour.

**ACKNOWLEDGMENTS**

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**REFERENCES**


Mini-symposium on Structures Incorporating FRP Composites under Impact/Blast Loading

Organizers:
Alex REMENNIKOV
Jingsi HUO
RESEARCH ON FRP ENHANCED STRUCTURES AND STRUCTURAL COMPONENTS SUBJECTED TO IMPACT LOADING

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ABSTRACT
This paper provides a summary on the research activities and research findings on FRP enhanced structures and structural components by the author’s research team in recent years. The research started with designing and establishment of several key experimental testing equipment, including large-size split Hopkinson pressure bars (SHPB), large-scale drop-weight machine, and a full-scale field testing facility for vehicle/structure collision. Fundamental tests on concrete cylinders and confined concrete cylinders under high-strain rate impacts were carried using the large size SHPB equipment. The research indicates that FRP confinement can enhance the impact resistance of the concrete and increase the energy dissipation capacity. The large-capacity drop-weight loading tests of FRP confined concrete or confined concrete filled steel tubes (CCFT) stub columns, reinforced concrete beams exhibited the enhancing effects of using external FRP wrappings. Dynamic loading tests on bond strength between CFRP and GFRP and concrete were also carried out, with findings exhibiting the dynamic effects. Finally, a large scale 3-story reinforced concrete frame model with 3 bay and 3 span was tested to examine the progressive collapse behavior due to sudden column failure by gas gun induced impacts.

KEYWORDS
FRP, RC structures, strengthening, interfacial bonding, confinement, collapse.

INTRODUCTION
Research needs of structures against blast and impact loads are becoming increasingly important in recent years, particularly after the 9-11 event in New York in 2001. Sponsored by several national and international research grants, the author has led a team at the Hunan University undertaking the efforts of experimental and analytical research on bridges and buildings subjected to potential manmade threats, such as blasts, impacts and fires, etc.

This paper attempts to summarize the research activities related to concrete structures under impact loading, at the Center for Integrated Protection Research of Engineering Structures (CIPRES), at the Hunan University, and the Nanjing Tech University.

EXPERIMENTAL FACILITY DEVELOPMENT
Since 2001, the author’s team has designed and developed several important testing facilities at the Hunan University. With the success at the Hunan University, the author’s research team was also given several contracting opportunities to design and manufacture similar testing equipment in China and elsewhere.

Large-scale SHPB
As shown in Figure 1, a SHPB (Split Hopkinson Pressure Bar, or Kolsky bar (Hopkinson 1914; Kolsky 1949; Nemat-Nasser et al. 1991; Forquin et al. 2008) contains a projectile, an input bar, an output bar, and a specimen placed between the input bar and the output bar. The projectile, propelled by gas-gun, impacts the input bar and transmits an elastic compression wave travelling in the input bar towards the specimen. Because the impedance of the specimen is less than that of the bars, an elastic tensile wave is reflected into the input bar and an elastic stress is transmitted into the output bar. Strain gages mounted on the input bar record the incident strain ε_1 and the reflected strain ε_R pulses, whereas strain gages mounted on the output bar record the transmitted strain ε_T.

The SHPB test technique contains two basic assumptions. One is that stress and strain fields in the specimen are uniform at any given instant. The other is that the stress fields are one-dimensional, uniform over the cross-section.
and in the axial direction. Once all three waves are recorded, the strain rate, strain and stress histories of specimen in uniaxial compression can be computed respectively from the following well know equations,

\[
\dot{\varepsilon}(t) = \frac{C_0}{L_s} \left[ \varepsilon_t(t) - \varepsilon_R(t) - \varepsilon_T(t) \right]
\]

(1)

\[
\varepsilon(t) = \frac{C_0}{L_s} \int \left[ \varepsilon_t(t) - \varepsilon_R(t) - \varepsilon_T(t) \right] dt
\]

(2)

\[
\sigma(t) = \frac{AE}{2A_s} \left[ \varepsilon_t(t) + \varepsilon_R(t) + \varepsilon_T(t) \right]
\]

(3)

where, \( \varepsilon_t(t), \varepsilon_R(t) \) and \( \varepsilon_T(t) \) are the surface strain-time histories induced by the incident, reflected and transmitted waves, respectively; \( C_0, A \) and \( E \) are the elastic wave velocity, cross-sectional area and Young’s modulus of the pressure bars respectively; and \( L_s \) and \( A_s \) are the length and cross-sectional area of the specimen.

The HNU-CIPRES is equipped with two SHPB shown in Figure 2, one with 74mm diameter and a temperature chamber up to 600°C, and the other with 100mm diameter. Huo et al. (2009) tested concrete filled steel tubes (CFT) under combined impact and elevated temperature using the 74mm diameter SHPB. At present, the author’s research team is also developing a 150 mm diameter SHPB at the Nanjing Tech.

**Drop-weight Tester**

The high-capacity drop-weight equipment at the Center for Integrated Protection Research of Engineering Structures (CIPRES), Hunan University, has a maximum drop height of up to 16 m, as shown in Figure 2. The hammer head weighs 124 to 200 kg depending on the testing purposes, and can hold additional thick steel plates to increase the hammer mass up to 1,000 kg. During testing, the hammer head is lifted up to predetermined height based on desired impact energy input and then the electrically controlled safety hook is released. Shortly before impacting, the data acquisition system and high-speed camera are triggered by using a laser sensing system. The impact forces of the hammer head is measured by a strain gauge based load cell and a piezoelectric load cell, whereas the forces at the specimen base can be checked also by piezoelectric film load cells.

**Field testing Facility for Vehicle/structure Collision**

Structures, particularly bridges and transportation facilities can be subjected to collisions, accidentally or intentionally, by vehicles in their service life. Different than the investigation of vehicular collisions by automobile industry, the emphasis in structural engineering is to study the structural behavior of structures, components and
facilities which receive the collisions or impacts. Another important research purpose is to study and develop structures or facilities to prevent or reduce the impact effects on structures by vehicular collisions. The vehicle/structure collision testing facility developed by the author’s team is shown in Figure 4 (Chen and Xiao et al. 2015).

The testing field has two tracks, one for collision truck which is guided by a rail with a length of 120m, and the other for pulling truck with shorter distance. During testing, the collision and pulling trucks are connected by a steel wire through a 1:2 pulley assembly system, which enables the reduction of the speed and travel distance of the pulling truck for desired collision speed. The wire pulling the collision truck can be relieved by a trigger system positioned 3.5m in front of the testing specimen, shown in Figure 4(b). Figure 4(c) shows a Dongfeng EQ-140 truck for testing with collision prevention bollards.

**FRP CONFINED CONCRETE**

**SHPB Tests of FRP Confined Concrete Cylinders**
To study the dynamic mechanical properties of the FRP-wrapped concrete under dynamic loading, the 100 mm diameter SHPB (Split Hopkinson Pressure Bar) is adopted to conduct the impact compression test of C40 (mix cubic strength of 40 MPa) concrete cylinder wrapped with 1 to 3 layers of basalt, carbon and glass fiber reinforced polymer (BFRP, CFRP and GFRP) wrapping under different strain rates, ranging from static to more than 200/s. The dynamic stress-strain relationships and dynamic increase factors (DIF) of different FRP confined concrete were obtained. Increasing the wrapping layers of FRP can increase the dynamic strength and deformability. Figure
5 exhibits the stress strain relationships of CFRP confined concrete and GFRP confined concrete with different wrapping layers. As compared in Figure 5, the effects of increasing wrapping layers for CFRP confined concrete is not as dramatic as for GFRP confined concrete. This might be due to the more brittle nature of CFRP compared with GFRP and BFRP.

Figure 5 Stress and strain relationships of FRP confined concrete under dynamic compression with an average strain rate of about 180/s: (a) confined concrete with 1 to 3 layers of CFRP wrappings; (b) confined concrete with 1 to 3 layers of GFRP wrappings.

Axial Impact Behavior of Confined Concrete Filled Tubes (CCFT)

Confined concrete tube (CCFT) is a new type of structural columns originally proposed by the author (Xiao et al. 2003, 2005). The impact tests on the CFT and carbon fiber reinforced polymer (CFRP) confined CFT stub columns (CCFT) under different impacting energy levels, were conducted using the HNU-CIPRES large-capacity drop-weight machine. The time history curves of impacting force and deformation time history curves as well as failure patterns were investigated. The results indicate that the failure patterns are related to the impact energy. Increasing the thickness of steel tube and providing additional transverse confinement by CFRP can enhance the impact-resistant behavior. The failure pattern of one of the CFRP CCFT specimens is shown in Figure 6(a). Finally, the dynamic analysis software ANSYS/LS-DYNA are used to simulate the impact behaviors of the CFT and CCFT specimens, and the simulation results are reasonable comparing with the test results. Figure 6(b) shows the comparison of impact force time histories obtained from the tests and the analysis (Xiao and Shen 2015).

DYNAMIC BOND STRENGTH BETWEEN FRP AND STEEL OR CONCRETE

Huo et al. (2016a, b) experimentally investigated the effect of the loading rate on the bond strength between the FRP and concrete. Twenty-three CFRP and Twenty-eight GFRP reinforced specimens with details shown in Figure 7(a) were tested using the drop-weight equipment, with the bending beam configuration shown in Figure 7(b). The investigation was conducted by changing the impact velocity, the bond width of FRP sheet and the number of FRP sheet layers. Figure 8 presents an example of strain distribution in CFRP at different load levels during a dynamic loading test. The test results show that the strain distribution gradient of the FRP sheets bonded to concrete
surface under impact loading was larger than under static loading, indicating the increase of the bond strength with increase of loading rate. It is also found that the effective bond length is reduced under dynamic loading. Equations for assessing the dynamic effective bond length and bond-slip model are also proposed by Huo et al. (2016a, b).

Figure 7 Dynamic bond testing of FRP and concrete: (a) specimen details; (b) test setup.

Figure 8 Strain distributions in CFRP for specimen C50-2-D600-1

FRP SHEAR ENHANCEMENT FOR RC BEAMS

The impact behavior of strengthened reinforced concrete (RC) beams with or without stirrups was experimentally investigated using the drop-weight equipment developed by the author’s research team. The impact test configuration is shown in Figure 9. The beams were retrofitted with FRP strips for shear reinforcement via a complete wrapping scheme. The CFRP reinforcement ratio was ranged from low to excessively high. The crack patterns and dynamic response including impact force, inertia force, reaction force, mid-span deflection, and CFRP strain were investigated. The CFRP contribution to impact resistance of RC beams was evaluated.

Figure 9 Drop weight impact test of FRP enhanced RC beam: (a) test setup; (b) specimen.

Bases on the observation and experimental data, the CFRP strengthening could change crack patterns and failure modes of RC beams under impact loading from severe shear failure without retrofit, to shear-flexural mode with shear-deficient CFRP ratio, to predominant flexural cracks at the critical ratio for shear reinforcement, and even to ductile flexure mode with excessive shear retrofitting amount, as shown in Figure 10.
The CFRP strengthening could significantly reduce the deflection of RC beams without stirrups by 80%. The increase of impact energy could linearly increase the deflection while the increase of impact velocity under same impact energy could decrease the deflection, as shown in Figure 11. The deflection distribution along the span presented irregular shapes if suffering from shear failure.

During the impact process, all dynamic forces could satisfy the dynamic force equilibrium. The inertia and reaction forces were activated later than impact force. At the initial phase, the impact force was mainly resisted by the inertia force, while the reaction force was balanced by the combination of impact and inertia force at the final low-frequency phase. If the CFRP reinforcement ratio was larger than the critical ratio for shear reinforcement based on the static design code of ACI 440.2R (2008), the RC beams without stirrups but retrofitted with CFRP complete wraps could avoid shear failure and merely suffer from flexural failure under ultimate impact loading.

![Figure 10 Crack patterns](image)

Figure 10 Crack patterns

![Figure 11 Maximum mid-span deflection](image)

Figure 11 Maximum mid-span deflection

**CFRP CABLE RETROFIT OF RC FRAME FOR COLLAPSE RESISTANCE**

A half-scale, three-story, two-bay and three-span reinforced concrete model frame (Figure 12a) retrofitted with carbon fiber reinforced polymer (CFRP) strip cables (Figures 12b and c) was tested under sudden removal of two side middle columns in the long direction on ground floor to study dynamic responses, resisting mechanism and CFRP retrofit effects. The CFRP strip cables were anchored to concrete slabs by specially designed anchorage (Figure 12c). The experimental results are compared with previous tests of the same frame without CFRP retrofit (Xiao et al. 2015). Figure 13 shows the displacement time histories of beam column joints immediately above the removed columns for the unretrofitted frame tested previously (Xiao et al. 2015) and the retrofitted frame subsequently (Liu et al. 2016). The comparison shows that the CFRP strip cables were effective in preventing the total collapse of the frame after the loss of columns. With additional CFRP strip cables, the frame structure sustained the applied gravity load as moment resisting frames without the transition to catenary action. To further demonstrate the effects, the CFRP strip cables were released, causing the frame structure to deflect downward and eventually collapse.

![Figure 12a](image)

(a) Without CFRP

![Figure 12b](image)

(b) Lower CFRP ratio

![Figure 12c](image)

(c) Critical CFRP ratio

![Figure 12d](image)

(d) Excessive CFRP ratio

Figure 12 CFRP retrofit of RC frame for collapse resistance
Figure 12 Testing of RC frame subjected to sudden removal of lower story columns: (a) plan view and column details; (b) elevation of RC frame and CFRP cable locations; (c) CFRP cable detail and anchorage.

Figure 13 Displacement time histories of RC frame with and without CFRP cable retrofit after sudden removal of lower story columns

CONCLUDING REMARKS

From the research reported in this paper, dynamic loading with high strain rates is shown to have significant effects on the behavior of FRP confined concrete columns and FRP enhanced reinforced concrete beams. Dynamic bond strength between FRP and concrete was shown to be increased with increased strain rates. A half-scale three story, three bay reinforced concrete frame structures strengthened using CFRP cables were demonstrated to be capable to avoid the progressive collapse due to the sudden removal of lower story columns.

Currently, the author’s team is continuing the research efforts on dynamic loading tests of reinforced and concrete filled steel tubular columns and the columns with FRP enhancement.
ACKNOWLEDGMENTS

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RC BEAMS STRENGTHENED WITH LONGITUDINAL AND U-WRAP FRP

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ABSTRACT

This study investigates the strengthening efficiency of modified reinforced concrete (RC) beams strengthened with longitudinal and U-wrap FRP. Two types of sections were studied, namely normal rectangular and modified sections. The RC beams were made of normal-strength concrete and then strengthened with different wrapping schemes, including longitudinal FRP layers and FRP U-wraps. The variables of this study are the thickness of FRP layers, the number of FRP U-wraps, and wrapping schemes. These specimens were tested under quasi-static loads until failure. The longitudinal strain and debonding mechanism of FRP are investigated. Experimental results have shown that the proposed modified section not only eliminates the stress concentration at the corners but also provides confinement to the longitudinal FRP layers. Thus, the modified beams showed higher load carrying capacity than that of normal-section beams even though the same amount of strengthening material had been used. Using FRP U-wraps together with modified section can maximize the effectiveness of longitudinal FRP layers. The strain of longitudinal FRP layers and U-wraps was also examined.

KEYWORDS

Fiber Reinforced Polymer, RC beams, U-wraps, strengthening, retrofitting.

INTRODUCTION

Fiber reinforced polymer (FRP) has been commonly used to strengthen reinforced concrete (RC) beams. The effectiveness of longitudinal FRP layers has been proven to significantly improve the flexural resistance of RC beams. However, it has been well documented that the premature debonding of FRP at the interface limits its efficiency (Quantrill et al. 1996; Garden et al. 1998; Ceroni 2010). There have been several methods to mitigate the premature debonding by using different anchors, for example, mechanical anchors, FRP fan anchors, and FRP U-wraps. Among these methods, using FRP U-wraps seems very attractive since it is a simple method and considerably delays the debonding of longitudinal FRP layers and thus enhances the flexural capacity (Buyukozturk et al. 2004; Ceroni 2010; Pham and Hao 2016). However, FRP U-wraps may fracture at the corners of the beam soffit because of the stress concentration even though the corners were rounded (Ceroni 2010). This study proposes a new method of modifying the beam soffit before wrapping with FRP. The proposed technique is expected to delay the premature debonding and eliminate the stress concentration and thus improve the strengthening effectiveness. This proposed strengthening technique is verified against the common strengthening technique through experimental tests carried out in the present study.

STRENGTHENING SCHEMES AND FRP DEBONDING

RC beams have been successfully strengthened with longitudinal FRP to increase their flexural resistance. In such cases, the soffits of beams were bonded with longitudinal FRP layers. However, the premature debonding of the longitudinal FRP strips limits the effectiveness of this strengthening technique. There are many causes leading to the debonding of longitudinal FRP strips as discussed in the study by Smith and Teng (2002). The most commonly reported debonding failure was at or near the FRP plate ends. This failure is likely due to high interfacial shear and normal stress (Smith and Teng 2002). In order to mitigate the debonding failure of FRP, transverse FRP wraps can be bonded to three sides of the beams, namely FRP U-wraps. The use of FRP U-wraps cannot generate force to against the peeling stress in the adhesive as shown in Figure 1, i.e., the tensile force in FRP does not contribute to resisting the peeling stress ($\sigma$) in the FRP U-wraps in the case of rectangular beams. The peeling stress is thus resisted by the tensile stress of the adhesive or concrete near the surface. On the contrary, if the beam soffit is modified to become an arc with a radius $r$, the FRP U-wraps can generate confining stresses which help to prevent the debonding of longitudinal FRP layers as shown in Figure 1. In such cases, the peeling stress is resisted by the
sum of the tensile stress in adhesive and the confining stress from the FRP U-wraps. The confining stress can be estimated as follows (Pham and Hadi 2014):

\[ \sigma = \frac{P}{r} \]  

where \( \sigma \) is the confining stress in the adhesive, \( P \) is the force in the FRP U-wrap, and \( r \) is the radius of the beam soffit.

![Debonding analysis of FRP strengthening](image)

**EXPERIMENTAL PROGRAM**

There are two beam groups with different sections, namely rectangular section and modified section. The beams with a modified section were cast in a special formwork including a rectangular steel formwork and curved polystyrene foam formwork. These beams were designed to fail in flexure so that their shear capacity is about four times of their flexural capacity (Table 1). For easier reference, the notation of the beams consists of three parts: The first part is N- and M- that states the shape of the section (Normal rectangular and Modified section). The second part indicates the wrapping arrangement in which L is for the number of the longitudinal layers while T stands for the number of transverse strips in half of a beam. For instance, L2T7 means this beam is wrapped with two layers of the longitudinal FRP layer and seven one-layer transverse FRP strips per the half beam.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Section</th>
<th>Longitudinal FRP (layers)</th>
<th>Transverse FRP (wraps)</th>
<th>Shear capacity (kN)</th>
<th>Flexural capacity (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>Rectangular</td>
<td>-</td>
<td>-</td>
<td>93</td>
<td>21</td>
</tr>
<tr>
<td>N-L1</td>
<td>Rectangular</td>
<td>1</td>
<td>-</td>
<td>93</td>
<td>23</td>
</tr>
<tr>
<td>N-L2T2</td>
<td>Rectangular</td>
<td>2</td>
<td>2</td>
<td>93</td>
<td>25</td>
</tr>
<tr>
<td>N-L2T7</td>
<td>Rectangular</td>
<td>2</td>
<td>7</td>
<td>123</td>
<td>25</td>
</tr>
<tr>
<td>M-L2T2</td>
<td>Modified</td>
<td>2</td>
<td>2</td>
<td>93</td>
<td>25</td>
</tr>
<tr>
<td>M-L2T7</td>
<td>Modified</td>
<td>2</td>
<td>7</td>
<td>93</td>
<td>25</td>
</tr>
</tbody>
</table>

- Not applicable

It is noted that these beams belong to the slender beam group as defined by MacGregor (2005) with the shear span ratio \((a/d)\) 4.52 in which \( a \) stands for the shear span and \( d \) is an effective depth of the beam. The dimensions of the rectangular beams were 150 mm in width, 250 mm in height, and 2200 mm in length. The modified section beams had the same length as the rectangular beams but the section was modified at the soffit which is an arc with a radius of 125 mm. The soffit of the beams was modified to postpone the premature debonding of the longitudinal FRP layers and reduce stress concentration at the soffit corners of FRP U-wraps. The details of the reinforcement are shown in Figure 2. The nominal tensile strength of deformed bars and plain bars are 500 MPa and 250 MPa, respectively. The ready-mixed concrete used to cast these beams had the compressive strength of 46 MPa at 28 day age.
The beams were bonded with a number of FRP layers at the beam soffit. In order to delay the debonding of FRP, FRP U-wraps were bonded vertically to three sides of the beams as shown in Figure 3. FRP was bonded to the substrate of concrete by the epoxy resin which has a tensile strength of 54 MPa, a tensile modulus of 2.8 GPa, and 3.4% tensile elongation (West System n.d. 2015). The adhesive used was a mixture of epoxy resin and hardener at 5:1 ratio. The FRP properties had been reported in previous studies (Pham et al. 2015a; Pham et al. 2015b). The CFRP used was 75 mm in width with a unidirectional fiber density of 340 g/m². The nominal thickness of FRP was 0.45 mm and the tensile strength was 1548 MPa. The average strain at the maximum tensile force and the average elastic modulus based on ASTM D3039 (2008) were 1.74% and 89 GPa, respectively.

Prior to bonding FRP to the beams, careful surface preparation was carried out to remove weak concrete. The concrete surface was roughened by using a pneumatic needle gun. An airgun was used to blow all dust and remove weak concrete particles. Acetone was used to clean the concrete surface. A primer was applied to the concrete surface before bonding with FRP. The epoxy curing time was maintained at least three days before testing.

A number of strain gauges were bonded to the soffit of the beams to monitor the longitudinal strains. The measurements are used to achieve: (1) the applied load versus FRP strain curves, (2) the distribution of FRP strain along the beams, and (3) the FRP strain at failure in which rupture or debonding of FRP could be expected. These strain gauges were placed at a spacing of 150 mm from one end of the beams to the midspan point as shown in Figure 3. Two strain gauges were also fixed on the side of each FRP U-wrap to monitor the FRP strain during the tests. The experiments involved testing six beams and measuring their responses to a monotonically increasing load. The beams were simply supported in a three point loading configuration using a roller and pin, creating an effective span of 1900 mm. The beams were tested using a hydraulic jack with a loading rate at 1 mm/min. The deflections of the beams were measured at different positions by linear variable differential transformers (LVDT).

EXPERIMENTAL RESULTS

Failure modes

As expected all beams failed under the flexure mode since they were designed to have relatively large shear resistance. The shear resistance of these beams is about 4 times higher than their flexural resistance. Vertical cracks appeared at the midspan of beams when the applied load reached about 19 kN. These cracks were observed for all beams including the reference beam and the strengthened beams as shown in Figure 4. As conventional RC beams failing in flexure, new vertical cracks were detected at positions closer to the supports. The widths of these cracks opened and their lengths developed from the soffit of the beams to the top. For the strengthened beams, when the applied load was substantial, the longitudinal FRP layers initially debonded at the midspan and the debonding extended to the beam ends. The longitudinal FRP layers were still able to carry tension stress as evidenced by the
increasing in longitudinal strain. Accordingly, the tension force in the longitudinal FRP layers horizontally pulled the vertical FRP U-wraps and caused the shear stress in the FRP U-wraps. When the FRP U-wraps ruptured, the beam failed due to the complete debonding of the longitudinal FRP layers as shown in Figs. 5b-c. Interestingly, the FRP U-wraps of Beams ML2T7 did not fail while rupture of the longitudinal FRP layers led the complete failure of this beam (Figure 5a).

![Beam R](image1)
![Beam NL1](image2)
![Beam NL2T2](image3)
![Beam ML2T2](image4)
![Beam NL2T7](image5)
![Beam ML2T7](image6)

**Figure 4 Failure modes of tested beams**

![Fractures of FRP U-wraps](image7)

(a) FRP rupture in Beam ML2T7
(b) Beams ML2T2 and NL2T2
(c) Beam NL2T7
(d) Concrete attached to FRP after failure

**Figure 5 Fractures of FRP U-wraps**

**Load-displacement curves**

The load – displacement curves of the reference beam and rectangular beams are presented in Figure 6. At the early stage of loading, the slope of the curves of these beams is almost the same, which means that the FRP layers had not sufficiently contributed to the capacity yet. When the applied load is greater than 50 kN, the applied load of strengthened beams still increases while the load of the reference beam remains unchanged and decreases afterward. Some discontinuities in the load – displacement curves of the strengthened beams were caused by debonding of the longitudinal FRP layers at the midspan. After FRP debonding stresses in these beams were redistributed and the applied load continued increasing while the debonding of the longitudinal FRP layers propagated to the supports. The experimental results of these beams are presented in Table 2. After the FRP layers fractured, these beams still could resist a load of the same level as the corresponding load of the reference beam at the same displacement. The capacity of Beam NL1 was increased by 14% while NL2T2 was enhanced 20%, as compared to the reference beam. The FRP U-wraps helped to postpone the debonding of longitudinal FRP layers. Especially, the capacity of Beam NL2T7 was enhanced by 59%, in which the longitudinal FRP layers debonded at the load of 66 kN but the FRP U-wraps prevented the propagation of the debonding. As a result, the capacity of Beam NL2T7 increased significantly until all FRP U-wraps at the midspan fractured.

The load – displacement curves of the modified-section beams are presented in Figure 7. As expected, the modified beams exhibited higher capacity than their counterpart in normal rectangular section group. The increase of the capacity in Beam ML2T2 was 34% compared to the reference beam. Interestingly, the capacity of Beam ML2T7 increased 87% even though this beam used the same amount of materials and less cross-sectional area as compared
to Beam NL2T7. It is recommended that the modified beams can be made by separately casting rectangular beams and curved segments before bonding them together by epoxy. The similar section modification method has been used in previous studies (Priestley and Seible 1995; Hadi et al. 2013; Pham et al. 2013). At the maximum load, the FRP U-wraps of Beam NL2T7 did not fracture while the longitudinal FRP layers ruptured. The curved soffit of this beam provides two fold advantages. It generated the normal stresses on the longitudinal FRP layer and reduced the curvature of the bottom corner of the section, which eliminated stress concentration of the FRP U-wraps as shown in Figure 1. As can be seen that $r$ is a finite value in the case of the curved beam section so that the normal stress $\sigma$ in FRP U-wraps provides confinement to the longitudinal FRP layer and then mitigates the debonding failure of the longitudinal FRP layer (Eq. 1). On the other hand, $r$ is infinite in the cases of the rectangular beams, thus the normal stress $\sigma$ in the FRP U-wrap does not prevent debonding of the longitudinal FRP layers. Meanwhile, the FRP U-wrap restrains the longitudinal deformation of the longitudinal FRP sheets and thus increases the shear resistance at the interface but not the normal stress resistance.

### Table 2. Experimental results of static tests

<table>
<thead>
<tr>
<th>Beams</th>
<th>Maximum load</th>
<th>Maximum Displacement</th>
<th>Debonding strain</th>
<th>Actual thickness of FRP</th>
<th>Load increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>54.5</td>
<td>51.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>NL1</td>
<td>62.1</td>
<td>50.2</td>
<td>0.59</td>
<td>1.1</td>
<td>14.0</td>
</tr>
<tr>
<td>NL2T2</td>
<td>65.2</td>
<td>41.2</td>
<td>0.56</td>
<td>1.7</td>
<td>19.6</td>
</tr>
<tr>
<td>NL2T7</td>
<td>86.6</td>
<td>51.6</td>
<td>0.60</td>
<td>2.1</td>
<td>58.9</td>
</tr>
<tr>
<td>ML2T2</td>
<td>73.1</td>
<td>56.1</td>
<td>0.67</td>
<td>1.9</td>
<td>34.1</td>
</tr>
<tr>
<td>ML2T7</td>
<td>101.8</td>
<td>30.2</td>
<td>1.09</td>
<td>2.1</td>
<td>86.8</td>
</tr>
</tbody>
</table>

*Not applicable
*as compared to the corresponding value of reference

**Figure 6 Load–displacement curves of rectangular beams**

**Figure 7 Load–displacement curves of modified beams**

### Debonding strain of FRP

Debonding in FRP strengthened concrete structures takes place in regions of high stress concentrations, which are commonly associated with discontinuities and the presence of cracks. Propagation path of debonding initiates from stress concentrations is affected by the material properties and their interface fracture properties (Buyukozturk et al. 2004). The FRP strain of Beam NL2T7 is used to analyze the debonding propagation and the debonding strain. It is noted that the locations of strain gauges are shown in Figure 3. Strain Gauges L3 and L4 located near the midspan of the beam increased steadily at the early stage of loading. After 950 seconds, these strain gauges suddenly dropped while Strain Gauges L1 and L2 were triggered at the same time. It means that debonding occurred and propagated to near the beam ends. Thus, the average value of Strain Gauges L3 and L4 is assumed as the debonding strain of this beam. In the meantime, the values of Strain Gauges TB1, TB2, TB3, and TB4 were very small (less than 0.16%) and thus are not presented. Interestingly, these FRP U-wraps fractured at the corners, implying the stress concentration at these positions was significantly high. The debonding strain of FRP of all the tested beams is summarized in Table 2.

In order to demonstrate the effectiveness of the proposed beam modification, the FRP strains of Beam ML2T7 is discussed. At about 1300 seconds, Strain Gauges L3 and L4 reached the average value of 1.09% while Strain Gauges L2 and L1 suddenly jumped to about 0.4%. This phenomenon was caused by the propagation of FRP
debonding from the midspan to the beam ends. In addition, some cracking sounds of the epoxy were heard at the same time. The beam then failed by rupture of the longitudinal FRP layers at the midspan (Figure 5a). The strain of the longitudinal FRP layers dropped while Strain Gauge TB4, which located on the FRP U-wrap at the midspan, suddenly jumped to a new value about 0.38%. It is obvious that the FRP strain of the proposed Beam ML2T7 was higher than the other beams and the stress concentration eliminated.

CONCLUSIONS

This study investigates the behaviour of RC concrete beams strengthened with FRP under quasi-static loads. The proposed beam modifications have been demonstrated successful in increasing the beam capacities. The proposed beam section modification delays the debonding of FRP and reduces the stress concentration at the corners. They significantly increased the beam capacities even though the same amount of materials is used as compared to its rectangular counterparts. Using FRP U-wraps is highly recommended to maximize the capability of longitudinal FRP layers. However, further studies are sought to quantify the effectiveness of FRP U-wraps on the premature debonding of longitudinal FRP and thus the flexural capacity of RC beams.

REFERENCES


DYNAMIC TENSILE PROPERTIES OF CARBON AND BASALT FIBRE REINFORCED POLYMER MATERIALS

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ABSTRACT

Fibre Reinforced Polymer (FRP) has been widely used in structural retrofitting and strengthening. FRP can be made of carbon fibre, basalt fibre or glass fibre. The structures strengthened by FRP might be subjected to blast and impact loading. To predict the structural performance of strengthened structure under dynamic loading, dynamic material properties of FRP need to be well understood. However, the study on the dynamic properties of FRP materials is limited in the literature. In this study, the strain rate effects on the dynamic material properties of both Carbon FRP (CFRP) and Basalt FRP (BFRP) were investigated. Quasi-static and dynamic uniaxial tensile tests of both the CFRP and BFRP were conducted. The failure modes and the testing results such as dynamic strength and failure strain of FRP at different strain rates were recorded. CFRP and BFRP material were compared in terms of the strength and failure strain. Based on the testing results, the empirical formulae of Dynamic Increase Factors (DIF) for CFRP and BFRP are derived to predict the strength increment at different strain rates.

KEYWORDS

Fibre Reinforced Polymer, dynamic tensile testing, strain rate effect, dynamic increase factor.

INTRODUCTION

Fibre Reinforced Polymer (FRP) has been widely used in retrofitting and strengthening existing structures as FRP strengthening is proved effective to enhance structure load-carrying capacity. FRP is made from polymer matrix reinforced with high strength fibres, such as carbon fibre, basalt fibre and glass fibre. During the service life, structures strengthened by FRP might be subjected to blast and impact loading. To predict the structural performance of strengthened structures under dynamic loading, dynamic material properties of FRP need to be well understood. However, there is a lack of understanding of the dynamic material properties of FRP for reliable predictions of the performances of the FRP strengthened structures subjected to dynamic loads at different loading rates.

In recent years, some studies have been conducted to examine the static and dynamic mechanical properties of CFRP and GFRP materials (Ou and Zhu 2015; Gilat et al. 2002; Al-Zubiady et al. 2013; Barre et al. 1996; Shokrieh and Omidi 2009). Nevertheless, contradicted testing results were reported on the dynamic properties of FRP material. For instance, for CFRP material Welsh and Harding (1985) tested T300 carbon/epoxy laminates at strain rate from 1.5x10^{-4}s^{-1} to 700s^{-1}. An increase in both the tensile strength and elastic modulus was reported. Zhao et al. (2007) and Chen et al. (2013) also carried out dynamic tests using SHPB technique on T300 carbon/epoxy composite respectively. Both testing results indicated that the tensile strength and the initial modulus increase with the strain rate. Recently, Al-Zubiady et al. (2013) tested CF130 carbon/epoxy laminates in the strain rate range of 0.000242s^{-1} to 87.4s^{-1}. The increased strain rate was found to result in increase in material tensile strength and modulus. However, some opposite conclusions on the effect of strain rate were given based on dynamic testing results on CFRP material. For example, with dynamic tests on T300 carbon/epoxy laminates, Hou and Ruiz (2000) observed insignificant strain rate effect on material tensile strength and modulus, while the shear properties were found to be strain rate dependent. Similar conclusions were made by Taniguchi et al. (2007) through their dynamic testing on type T700 carbon/epoxy laminates. In general, there is no consensus on the significance or even existence of strain rate effect on CFRP material, and the degree of its sensitivity. For BFRP, available literatures on its dynamic material properties are very limited. Therefore, there is a need to properly understand dynamic material properties of CFRP and BFRP for better analysis and design of FRP strengthened structures against dynamic loads.
In this study, the strain rate effect on the tensile properties of carbon and basalt fibre reinforced polymers was investigated. A series of quasi-static and dynamic tensile tests were carried out. The strain rates of the dynamic testing in this study were up to $237 \text{s}^{-1}$ and $259 \text{s}^{-1}$ for CFRP and BFRP, respectively. The failure modes and the testing results were recorded and compared. Empirical formulae for the CFRP and BFRP tensile material properties are proposed.

**EXPERIMENTAL PROGRAM**

**Testing specimens**

The carbon fibre tested in this study is SikaWrap®-230C, which is a woven unidirectional carbon fibre fabric designed for structural strengthening applications. It is a mid-range strength CFRP material in its family. The fibre density is $1.8 \text{g/cm}^3$ and the fabric sheet has a nominal thickness of $0.131 \text{mm}$. The epoxy resin used is Sikadur®-330 whose tensile strength and elastic modulus are $30 \text{MPa}$ and $4.5 \text{GPa}$, respectively. Unidirectional basalt fibre (named BJ30) with a unit weight of $300 \text{g/m}^2$ and a nominal thickness of $0.12 \text{mm}$ was tested. CFRP and BFRP laminate specimens were prepared by using wet lay-up process as per the guidelines ASTM 3039-14 and ASTM 7565-10. The thicknesses of the CFRP and BFRP laminates are about $0.5 \text{mm}$ and $0.7 \text{mm}$, respectively. The laminate sheets were sliced into $25 \text{mm}$ wide strips. It is imperative that the fibres run parallel to the direction of loading. The tabs were glued onto the specimens to avoid any possible damage when gripping the specimen. The gauge length of the dynamic specimen is $50 \text{ mm}$. Figure 1 shows the dimensions of the specimens for quasi-static and dynamic testing.

![Figure 1](image)

**Testing setup**

The quasi-static and dynamic tensile tests were performed by using hydraulic testing system and INSTRON® VHS 160/100-20 testing system, respectively (Figure 2). The INSTRON® machine utilizes servo-hydraulic and control technology and is capable of providing a controlled testing velocity of up to $25 \text{ m/s}$ under open loop control and $1 \text{ mm/s}$ to $1 \text{ m/s}$ under closed loop control. A maximum impact load of $100 \text{ kN}$ can be achieved in the testing system. In the dynamic tests, actuator speeds were varied from $0.5 \text{ m/s}$ to $20 \text{ m/s}$.

The INSTRON® VHS system comprises a fast jaw grip, which accelerates in the direction of tension till the designed testing velocity is achieved. A pre-set wedge is kicked out to release the sprung grips which firmly grab the upper tap and pull the specimen at the designed testing velocity until its fracture. A piezo load cell is installed below the bottom grip head to monitor the force that the specimen experienced. An accelerometer was built on the fast jaw to measure its acceleration. In the tests, an extensometer in the fast jaw was utilized to track the stroke of the actuator. A strain gauge was glued to the centre of each specimen to measure its strain. All signals of the above transducers were connected to a data acquisition system with a sampling frequency of $65 \text{ kHz}$. A high-speed camera was installed to monitor the dynamic fracture process of the FRP laminates, which was triggered by a TTL pulse from the testing system. A 2000 w halogen light was used to provide the lighting for high-speed filming. The filming frequency was set to $30,000 \text{ fps}$.
TESTING RESULTS

Failure modes

The typical failure modes for CFRP and BFRP are shown in Figure 3. Under quasi-static state, coupon breakages occurred in the central part of the specimen while some breakages were found around the grip area due to stress concentration or unavoidable loading eccentricity. Under dynamic tension, the failure mode of CFRP specimen differed from that under quasi-static state. Two diagonal cracks were formed instead of a single crack at gauge centre. The CFRP specimen was eventually split into three segments by the two diagonal cracks. More energy was therefore consumed, which led to higher material strength. For the BFRP specimen, fibre fracture was observed around the grip area and the delamination of fibres was found across the whole gauge section.

Stress-strain curves

The typical stress-strain curves of CFRP and BFRP at different strain rates are shown in Figure 4. Under quasi-static state, the BFRP specimens behaved as linear elastic material until fibre fracture. An averaged fibre strength, failure strain and elastic modulus are calculated as 1642.2MPa, 0.021 and 77.9GPa, respectively. As can be observed, the CFRP specimens also behaved as a linear elastic material until the point of failure. At a strain rate of 8x10^-5 s^-1, the fibre strength of 2953MPa was measured. As loading rate increases, the tensile strength increased marginally. At a strain rate of 8x10^-4 s^-1, the fibre strength increased to 3062MPa. The corresponding failure strain reduces marginally from 1.09% to about 1.06%. Nevertheless, the increase in specimen strength is not continuous and stable with the increase of strain rate. When strain rates go up to 4x10^-3 s^-1 and then further to 7.7x10^-3 s^-1, there is no significant increase in coupon strength and fibre strength. As shown in Figure 4a, under dynamic tension the basalt fibre shows nearly linear elastic behaviour until failure. The stress-strain curves of the specimens indicate an increase in tensile strength and failure strain with the strain rate. When the strain rate increases to 259s^-1 (Figure 4a), the fibre strength of BFRP goes up to 3383.3 MPa, with an increment of 106.0%. The corresponding failure strain is observed to be around 0.03, indicating an enhancement on the dynamic failure strain as compared to the quasi-static failure strain of around 0.021. Under dynamic loading, the CFRP also showed elastic behaviour. Stress increases almost linearly with strain at the beginning. When the strain approaches failure strain, the stiffness increases slightly indicating minor non-linear behaviour. The plot also reflects that the response of CFRP laminates is strain rate dependent. At a strain rate of 11s^-1, the fibre strength of 2985 MPa is reached, which is similar to that under quasi-static loading. As strain rate increases, CFRP fibre strength increases. At a strain rate of 185s^-1, the
fibre strength rises to about 5009 MPa, which reflects an increment of 63%. When strain rate further increases to 233 s\(^{-1}\), the fibre strength goes up to 5506 MPa, indicating an 80% increment.

Figure 4 Stress-strain curves of BFRP and CFRP at various strain rates

**ANALYSIS AND DISCUSSIONS**

**Strain rate effect on tensile strength**

Figure 5 plots the tensile strengths of basalt fibre and carbon fibre versus strain rate. The tensile strengths of fibres are calculated by only considering the thickness of basalt fibre (i.e. 0.12 mm) and carbon fibre (i.e. 0.131 mm). As shown in Figure 5, the fibre strengths of both basalt fibre and carbon fibre in the current tests show consistent behaviour with strain rate. For CFRP, the fibre strength does not show much variation with strain rate at low-strain rates. Significant increase in fibre strength can be found when strain rate is over approximately 50 s\(^{-1}\), which rises quickly with strain rate. The strength of basalt fibre is in general lower than that of carbon fibre because of different fibre properties. Strong strain rate dependency can be observed in Figure 5 that the BFRP fibre strength increased from about 1800 MPa at quasi-static state to about 2000 MPa at a strain rate of around 10 s\(^{-1}\). When the strain rate goes above 120 s\(^{-1}\), the increase in BFRP fibre strength rises much quicker with strain rate.

Figure 5 Fibre tensile strength vs. strain rate

Figure 6 Fibre failure strain vs. strain rate

**Strain rate effect on failure strain**

Figure 6 shows the failure strains of the CFRP and BFRP versus strain rate. The quasi-static failure strain of the BFRP is around 0.021. The failure strain does not show prominent strain rate sensitivity when the strain rate is below 120 s\(^{-1}\). When the strain rate is over 120 s\(^{-1}\), the failure strain goes up quickly with strain rate. When the strain rate is 259 s\(^{-1}\), the failure strain is 0.03, with an increase of 42.9% as compared to the quasi-static failure strain. The failure strain of CFRP is much lower than that of BFRP fibre. In the quasi-static and low-strain rate range the CFRP fibre fractures at strain around 0.01. Similarly to the relationship between the strength and strain rate presented above, the failure strain in the low-strain rate range does not show much correlation with strain rate. But
when the strain rate increases beyond 50s$^{-1}$, apparent higher failure strain can be observed, which also increases with strain rate. When the strain rate is around 50s$^{-1}$ the failure strain increases to about 0.011 with an increase of 10% comparing with the failure strain at quasi-static state. When the strain rate reaches to around 200s$^{-1}$, the failure strain rises to about 0.014, indicating a 40% increase.

**Strain rate effect on fibre modulus**

Figure 7 shows the relationship of fibre elastic modulus versus strain rate. Similar to failure strain, the fibre elastic modulus of BFRP shows limited sensitivity to strain rate in the low strain rate range. When the strain rate goes above 120s$^{-1}$, the fibre elastic modulus increases remarkably with the strain rate. When the strain rate is up to 259s$^{-1}$, the elastic modulus increases from 77.9GPa to 111.7GPa, with an increment of 43.4%. The modulus of CFRP is much larger than that of BFRP. For instance, at quasi-static state the modulus of CFRP is over 200GPa, while that of BFRP was less than 80GPa. The fibre modulus of CFRP also shows strain rate dependency. In the low-strain rate region carbon fibre modulus appears to be steady around 200GPa with the increase of strain rate. Clear dynamic increase effect can be found in the high-strain rate range. For instance, at strain rate of 20s$^{-1}$, an averaged fibre modulus of 255GPa was measured. As strain rate increases to about 100s$^{-1}$, the fibre modulus increases to around 305GPa with an increment of 50% as compared with quasi-static state. When the strain rate increased to about 200s$^{-1}$, the averaged fibre modulus increases to about 375GPa indicating a further 23% increase.

**Empirical formula of DIF for fibre tensile strength**

Dynamic increase factor (DIF), as a ratio of dynamic strength over static strength, can be employed to describe the increase in material strength at different strain rates. The measured fibre strengths of CFRP and BFRP in the current study are normalized against their averaged fibre strength as quasi-static state to derive the DIF for fibre strength. Figure 8 shows the DIF for both CFRP and BFRP fibre strengths versus the testing strain rates. The DIF for BFRP is up to 2.09 at strain rate of 256 s$^{-1}$ and the DIF for CFRP is up to 1.83 at strain rate of 220 s$^{-1}$. A bi-linear trend line can be introduced to depict the trend of the DIF and strain rate relationships for both CFRP and BFRP.

The empirical equations to predict DIF of CFRP fibre strength can be given as follows:

\[
\text{DIF}_{\text{CFRP}} = 0.0001 \times \log_{10}(\dot{\varepsilon}) + 1.003 \quad \dot{\varepsilon} < 50 \text{s}^{-1}
\]

\[
\text{DIF}_{\text{CFRP}} = 1.095 \times \log_{10}(\dot{\varepsilon}) - 0.857 \quad \dot{\varepsilon} \geq 50 \text{s}^{-1}
\]

The empirical equations to predict DIF of BFRP fibre strength can be given as follows:

\[
\text{DIF}_{\text{BFRP}} = 0.0456 \log_{10}(\dot{\varepsilon}) + 1.180 \quad \dot{\varepsilon} < 120 \text{s}^{-1}
\]

\[
\text{DIF}_{\text{BFRP}} = 1.7 \log_{10}(\dot{\varepsilon}) - 2.25 \quad \dot{\varepsilon} \geq 120 \text{s}^{-1}
\]

where DIF$_{\text{CFRP}}$ and DIF$_{\text{BFRP}}$ stand for the DIF of CFRP and BFRP, respectively. $\dot{\varepsilon}$ is strain rate.

![Figure 7 Fibre elastic modulus vs. strain rate](image1)

![Figure 8 DIF of fibre strength vs. strain rate](image2)
CONCLUSIONS

In this study, experimental tests were conducted to investigate the quasi-static and dynamic properties of carbon and basalt fibres. The strain rates of the dynamic testing in this study are up to 237 s^{-1} for CFRP and 259 s^{-1} for BFRP. Stress-strain relationships of CFRP and BFRP at various strain rates were measured. It was found that the dynamic material properties of both carbon fibre and basalt fibre are sensitive to the strain rate. The fibre properties such as tensile strength, failure strain and fibre modulus increase significantly with the strain rate when the strain rate is over 50 s^{-1} for CFRP and 120 s^{-1} for BFRP. The failure modes at different strain rates were captured by high-speed camera to analysis the strain rate effect. Multiple fracture, diagonal cracks and delamination of the specimens were observed in the dynamic testing. Empirical formulae of DIF for the CFRP and BFRP fibre strength with respect to the strain rate were derived from the testing data.

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REFERENCES


IMPACT BEHAVIOR OF UHPFRC PANELS UNDER MODERATE VELOCITY IMPACT LOADING

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ABSTRACT

The present study is intended to investigate impact behavior of Ultra High Performance Fibre Reinforced Concrete (UHPFRC) plates by conducting moderate velocity impact tests. In a series of tests, a steel hemispherical projectile with a mass of 8.3kg collided to 6cm-12cm-thick UHPFRC panels, in which steel fibres were mixed with pre-mix high strength mortar. A range of impact velocity was around 40m/s. Test results revealed that the UHPFRC panels have significantly suppressed the scabbing and perforation failure compared to plain concrete plates. Based upon test data such as impact load-time history, reaction load-time history, strain response on the back surface and failure behaviour captured by a high speed video camera, a failure mechanism of the UHPFRC panel was discussed.

KEYWORDS

UHPFRC, moderate velocity impact, protective design

INTRODUCTION

In recent years, tornado and volcanic activity, which have been affected by climate change and crustal movement, has increased and made awareness on the protection of key structures or shelters. The velocity of a tornado missile is set to be 51 m/s (about 184 km/h) in the assessment guide for tornado effect on nuclear power plants (Nuclear Regulation Authority of Japan 2013). In the volcanic eruption, volcanic missiles were flown to approximately 2 km from the crater. The velocity of the volcanic missile with 10cm diameter is assumed approximately 100 m/s (360 km/h) (Cabinet Office of Japan 2015). While several studies have investigated the impact resistant design of concrete structures against the low to moderate velocity impacts described above in recent years, Ultra High Performance Fibre Reinforced Concrete (UHPFRC) has been developed and utilized as a structural member (e.g. Tanaka et al. 2002). The UHPFRC has excellent strength and durability characteristics compared to plain concrete. Hence, the UHPFRC can be used in protective structures against the impact loads. In regard to the impact resistance of the UHPFRC, past studies have focused on their impact resistant capabilities in low-velocity impact whose velocity is less than 10 m/s (e.g. Fujikake et al. 2006) and high velocity impact whose velocity is 100 m/s-500 m/s (e.g. Soe et al. 2013; Sovjak et al. 2013). On the other hand, few studies on the low to moderate-velocity impact of the tornado and volcanic missiles were done.

This study aims to investigate impact behavior of UHPFRC panels experimentally, subjected to low to moderate-velocity impact. Using low or moderate-velocity, impact tests on 18cm thick plain concrete plates and 6cm-12cm thick UHPFRC panels were conducted. Based upon test data such as impact load-time history, reaction load-time history, strain response on the back surface and failure behaviour captured by a high speed video camera, a failure mechanism of UHPFRC panels was discussed.

EXPERIMENTAL SETUP

Moderate-velocity impact test machine

A schematic diagram of a projectile launching system is depicted in Figure 1. This system consisted of an air compressor of an air booster, an air chamber and accelerating tube. This system was able to launch a projectile with a mass of between 4kg and 10kg, and with an impact velocity of between 10m/s and 80m/s, by adjusting
compressed air pressure. Impact velocity of the projectile was measured by two laser type speed sensors with resolution of 1cm/s.

**Projectile and Specimen**

In a series of tests, an 8.3kg spherical steel projectile with a diameter of 90mm was used as shown in Photo 1, and this mass is identical to the steel missile mass prescribed in the assessment guide for tornado effect on nuclear power plants (Nuclear Regulation Authority, Japan. 2013). An outlook of a plain concrete plate is shown in Photo 2. Dimension of the plain concrete plate was 1.2m in length and height and 18cm in thickness. The compressive strength of the plain concrete was 33.9N/mm$^2$ on average. Dimension in length and height of UHPFRC panels used in this study have the same dimension as the plain concrete plate and thicknesses were 6cm, 9cm and 12cm. The mix proportion of the UHPFRC followed the guidelines of UHPFRC construction design (draft) (Japan Society of Civil Engineers 2004) and mixing ratio of high-strength steel fibre, whose tensile strength $P_u=2,800$N/mm$^2$, diameter $\phi=0.2$mm, length $L=15$ mm, is 2 Vol%. The average compressive and tensile strengths of the UHPFRC were 216N/mm$^2$ and 24.6 N/mm$^2$ respectively, by standard heat curing of 48 hours in 90°C steam. Table 1 shows the mix proportion of the plain concrete and UHPFRC. The static punching shear strength of the UHPFRC of 6cm, 9cm, and 12cm thick panels were 249kN, 448kN and 697kN respectively.

**Measurement items and testing case**

Measurement items were displacement, velocity of the projectile, impact and reaction loads as well as penetration depth, diameter of spalling and strain response of back surface. Impact velocity varied from 37.9 m/s to 43.5 m/s. In general, load cell is used for measurement of an impact load, but a wire of the load cell seemed to be taken broken in this test condition. Hence, the impact load was estimated by multiplying the acceleration by the projectile mass of 8.3kg which was calculated by differentiating the velocity-time history of the projectile captured by a high-speed video camera with a frame rate of 50,000fps. In order to get rid of high-frequency components in the acceleration-time history, a moving average filter has processed at cut-off frequency of 3 kHz. Figure 2 shows the location of measurement items. Penetration depth was the deepest position of the dent from the impact surface. The spalling diameter was measured in four directions. Figure 3 shows the location of strain gauges on the back surface of the specimens. To measure the collision time on the impact surface, a 60 mm strain gauge was set in the center of impact surface. To investigate the failure response on the back surface, five 60mm strain gauges were attached in a 100 mm interval from the center of the back surface. The location of load cells (response frequency: 30 kHz) for measurement of the reaction force is shown in Figure 4. The plain concrete plates were installed with

![Figure 1 Outline of a moderate velocity impact test machine](image)

![Photo 1 Steel projectile](image)

![Photo 2 Plain concrete specimen](image)
a 4 sides support frame assuming that they were used as slab members in practice. On the other hand, the UHPFRC panels were installed with a 2 sides support frame because they were used generally as precast members (e.g. Musha et al. 2013).

RESULTS AND DISCUSSIONS

Failure state

Table 2 and Photo 3 summarize test results and damages of the specimens respectively. Case name is listed using material type "PLAIN" or "UHPFRC" with thickness (mm) and case number. The failure mode of PLAIN-180-1 at an impact velocity of 37.8 m/s was spalling with the penetration depth of 12.3mm, where thin diagonal and radial cracks were observed in the cross section and the back surface, respectively. The failure mode of PLAIN-180-2 was spalling with the penetration depth of 33.1mm at an impact velocity of 41.4 m/s, as diagonal and radial cracks developed as the impact velocity increased. Note that since some fragments in the back surface were detached during cutting of the specimen, the diagonal crack looked greater than the actual state. The failure mode of PLAIN-180-3 at an impact velocity of 43.5m/s was judged the limit of perforation because a clear hole was generated, but the projectile didn’t pass through the specimen. From the failure states of the specimens, the velocity which yields the scabbing for the 18cm-thick plain concrete plates was assumed between 41m/s and 43m/s. In all cases, diagonal cracks in the cross section of the specimen looked like punching shear failure. Referring to several definitions of local damage or overall response proposed by the literatures, a further investigation is needed to examine the failure mechanism precisely.

The failure mode of UHPFRC-60-1 was perforation limit with spalling diameter of 85 mm which was slightly smaller than the diameter of the projectile. The UHPFRC-90 specimens at an identical impact velocity of 41.4m/s showed scabbing (UHPFRC-90-1) and was spalling (UHPFRC-90-2), respectively. This fact indicates that the thickness of 9cm of the UHPFRC panel is the scabbing limit at an impact velocity of 42m/s, when using an 8.3kg projectile. Although the boundary condition of the UHPFRC panels was different from that of the plain concrete plates, both failure states resemble each other and the scabbing limit thickness of the UHPFRC panels was reduced approximately by 50% compared to the plain concrete plates. This finding indicates the same trend of the test result conducted by Musha et al (e.g. Musha et al. 2013). Also, diameters of spalling, scabbing and perforation, were smaller than those of the plain concrete plates. In the cross section of UHPFRC-90-1, many diagonal cracks
were dispersed from the center of the impact area due to the cross-linking effect of the fibre reinforced concrete (e.g. Fujikake et al. 2006). The failure mode of UHPFRC-120 was spalling with the spall diameter of 112 mm. whereas the spalling diameter was longer than the other cases, occurrence of the diagonal cracks in the cross section has been greatly restrained. The experimental results indicate that UHPFRC panels subjected to low to moderate-velocity impact of a hard projectile have restrained the local failure significantly compared to plain concrete plates.

**Table 2 Test cases and dimension of local damage**

<table>
<thead>
<tr>
<th>No.</th>
<th>Test case</th>
<th>Velocity (m/s)</th>
<th>Damage mode</th>
<th>Depth of penetration</th>
<th>Diameter of spalling</th>
<th>Diameter of scabbing</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>PLAIN-180-1</td>
<td>37.9</td>
<td>Spalling</td>
<td>12.3</td>
<td>65.7</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>PLAIN-180-2</td>
<td>41.4</td>
<td>Spalling</td>
<td>33.1</td>
<td>201</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>PLAIN-180-3</td>
<td>43.5</td>
<td>Perforation limit</td>
<td>-</td>
<td>238</td>
<td>817</td>
</tr>
<tr>
<td>4</td>
<td>UHPFRC-60-1</td>
<td>41.9</td>
<td>Perforation limit</td>
<td>-</td>
<td>87.3</td>
<td>257</td>
</tr>
<tr>
<td>5</td>
<td>UHPFRC-60-2</td>
<td>41.6</td>
<td>Perforation limit</td>
<td>-</td>
<td>81.8</td>
<td>235</td>
</tr>
<tr>
<td>6</td>
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<td>Scabbing</td>
<td>10</td>
<td>57.3</td>
<td>392</td>
</tr>
<tr>
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<td>Spalling</td>
<td>15.6</td>
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<td>-</td>
</tr>
<tr>
<td>8</td>
<td>UHPFRC-120</td>
<td>41.6</td>
<td>Spalling</td>
<td>9.4</td>
<td>112</td>
<td>-</td>
</tr>
</tbody>
</table>

**Figure 5** shows the displacement-time, impact velocity-time and impact load-time histories of the projectile of PLAIN-180-3, which are recorded by the high-speed video camera, and reaction load-time history is also shown in the figure. It can be seen from Figure 5(a) that the displacement increases gradually with time and comes to 38mm at t=1.8ms, which in accordance with the penetration depth of 35mm estimated from Photo 3. Figure 5(b) indicates that the initial impact velocity is 48m/s, which is 5m/s higher than that obtained with the velocity sensor. This discrepancy may be caused by an acceleration of the projectile after passing the velocity sensor. Figure 5(c) indicates that peak impact load shows 490kN at t=0.7ms. Subsequently, the impact load decreased and oscillation with high frequency components was induced during t=1.4ms-1.8ms, which was caused due to a cloud of dust by
the collision of projectile. Reaction load arose at t=0.8ms and came to 200kN at t=2.2ms. Figures 6 and 7 show the displacement-time, impact velocity-time and impact load-time histories of the projectile recorded by the high-speed video camera and the reaction load-time history of the UHPFRC-60-2 and UHPFRC-90-1, respectively. Figure 6(b) indicates that after maximum displacement at t=3ms, the impact velocity of the projectile showed almost zero, namely, most of the kinetic energy of the projectile was consumed in the penetration process. Figure 6(c) indicates that the impact load gradually increased to approximately 400kN at t=0.3ms. Subsequently, after the impact load dropped sharply, the impact load came to 200kN at t=0.7ms with duration time of 3.0ms. The duration of impact load was 1.5 times longer than that of PLAIN-180-3, because the failure developed gradually by the cross-linking effect. With regard to the UHPFRC-90-1, Figure 7 indicates that the maximum displacement was 22mm at t=1.0ms which was 2 times of measured penetration depth of 10 mm and the impact velocity showed zero at t=1.2ms. The discrepancy of the penetration depth between data analysis and observation might be caused by the overall deformation of the specimen in addition to the local failure. The maximum impact load as shown in Figure 7(c) was 700kN which is twice that of UHPFRC-60-2 and the duration time of the impact load was 1.4ms. The shorter duration time compared to plain concrete plates was caused by the high stiffness of the UHPFRC-90-1 panels. The impulse of UHPFRC-90-1 calculated by integrating the impact load time-history was 371kN * ms which is agreed well with the initial momentum of the projectile. The reaction load was generated at t=1ms and the impact load decrease to zero at the same time. This fact indicates that the overall deformation of UHPFRC panels arose at the end of the contact between the projectile and the UHPFRC panels.

Strain in the back surface and failure behavior of specimen

Figures 8 and Photo 4 show the strain-time history on the back surface and pictures of the failure behavior by the high-speed video camera in the case of UHPFRC-60-2, respectively. Figure 8 indicate that strain gauge B1 showed extremely large tensile strain at t=0.2ms because the strain gauge was disconnected by the penetration of the
From the photos of high-speed camera as shown in Photo 4, where the origin time was determined by recognizing a crack in the picture, the circumferential crack was developed around the center of specimen (gauge B1-B2) at t'=0.12ms. By comparing strain-time history with the pictures, the tensile strain of gauge B2 as shown in Figure 8 indicates the strain generated by the local deformation, and it finally arose sharply with the progress of the scabbing. At t'=0.66ms, the fragment of the scabbing was broken into pieces and debris inside and outside the specimen. By comparing the impact load-time and reaction load time-histories as shown in Figure 7(c), the crack and fragment in the back surface occurred at the initial phase of the reaction load and the local damage prior to the overall response.

CONCLUSIONS

This study investigated impact behavior of UHPFRC panels experimentally, subjected to low to moderate-velocity impact. The results of this study are summarized as follows.

(1) Moderate-velocity impact tests of plain concrete plates and UHPFRC panels were conducted. As a result, failure states of the UHPFRC panels, which had almost half thickness of the plain concrete plates, were similar to the plain concrete plates at an impact velocity of 42m/s. This fact indicates that the UHPFRC panels had a superior impact resistance capability.

(2) By comparing an impact load-time, reaction load-time, strain-time histories and failure behavior, the 6-9cm thick UHPFRC panels were failed by the local damage rather than the overall response arose.

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IMPACT RESISTANCE OF ULTRA-HIGH STRENGTH CONCRETE BEAMS WITH FRP REINFORCEMENT

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ABSTRACT
Composite materials, including Fibre Reinforced Polymer (FRP) bars, have been used for decades in the structural and civil engineering sectors over traditional steel reinforcement. The main reasons for this are that FRP composites possess a number of advantages. They are non-corrosive, non-conductive, and lightweight and possess high longitudinal tensile strength. This paper presents the results of an experimental investigation into the effects of the use of glass FRP (GFRP) bars as internal reinforcement on the behaviour of concrete beams with high strength concrete (HSC) and ultra-high strength concrete (UHSC). Both static and dynamic (impact) behaviours of the beam have been investigated. Twelve GFRP reinforced concrete (RC) beams were designed, cast and tested. Six GFRP RC beams were tested under static loading (three point bending) to examine the failure modes, load carrying capacity, deflection and energy absorption capacities. The other six GFRP RC beams were tested under impact loading using a drop hammer apparatus at various levels of impact energy. It was found that the use of UHSC in conjunction with larger amounts of tensile reinforcement showed higher levels of post-cracking bending stiffness. GFRP RC beams under static loading displayed a flexural response at failure. The GFRP RC beams under impact loading displayed a dynamic punching shear failure response at various levels of impact energy.

KEYWORDS
FRP, RC beams, High strength concrete, Flexure, Impact.

INTRODUCTION
Experimental studies carried out have investigated the impact behaviour of RC beams reinforced with conventional steel reinforcement (Chen and May 2009, Fujukake et al. 2009, Saatci and Vecchio 2009). There are three types of responses that a RC beam can be subjected to – local response, global response or a combination of both. Localised failure modes of RC beams under impact have been described as being scabbing, which results in spalling and detachment of the tensile concrete cover, penetration, diagonal shear cracking around contact zone. This type of response is typically referred to as a shear “plug” type, even for flexural-critical RC beams (Saatci and Vecchio 2009) or localised dynamic punching shear failure (Kishi et al. 2002, Zhang et al. 2005) which has shown to occur at higher velocities of impact (Ohnuma et al. 1985). This type of response results in the majority of energy from the impact being dissipated around the impact area. A global response represents the bending and deformation response of the RC beams under impact. The behaviour of RC beams under impact loading has been described as a combination of both local and global response (bending and deformation). However, the global response has been documented as the main concern for RC beams subjected to impact loading (Hughes and Beeby 1982). The influence of different parameters including shear mechanisms, impact velocity, cracking response, impact energy and comparisons between static and impact failure modes were investigated in the literature. Also, most of the previous studies were limited to normal strength concrete (NSC). Only a limited number of studies have examined the impact response of high strength concrete beams reinforced with conventional steel reinforcement (Ágárdh et al. 1999). Although the behaviour of steel RC beams under impact loading were extensively studied, limited attention has been focused on experimentally investigating the impact response of concrete beams reinforced with GFRP reinforcement bars (Goldston et al. 2016). Goldston et al. (2016) reported that flexural-critical GFRP RC beams under impact loading displayed a shear “plug” type of failure, indicating the importance of shear mechanisms. Also, higher strength concrete and larger amounts of reinforcement, fewer inclined shear cracks were present along the surface of the GFRP RC beams (Goldston et al. 2016). However, there have not been any studies so far addressing the impact behaviour of ultra-high strength concrete (UHSC) beams reinforced with GFRP bars.
EXPERIMENTAL PROGRAM
Details of GFRP RC Beams

A total of twelve simply supported GFRP RC beams (2400 mm long, 100 mm wide and 150 mm deep) were constructed and experimentally tested under static loading and impact loading. The experimental program was divided into two series. The first series consisted of six GFRP RC beams tested under static loading (S) (three-point bending) to further investigate the influence of GFRP reinforcement bars on the flexural behaviour of beams. The test variables were the longitudinal reinforcement ratio and concrete strength. Three beams were cast with concrete of 80 MPa nominal concrete compressive strength with three beams cast with concrete of 120 MPa nominal compressive strength. At time of testing, concrete strength was measured as 97 MPa and 116 MPa. The parameters investigated were load-deflection behaviour, failure mode, energy absorption and strain in the concrete and GFRP reinforcement bars. The second series consisted of six beams tested under impact loading to investigate the dynamic response of UHSC GFRP RC beams. The six GFRP RC beams under impact loading (I) were cast with 120 MPa nominal compressive strength. Three beams had a reinforcement ratio of 1.0% and three with a reinforcement ratio of 2.0%. The GFRP RC beams were subjected to three different heights for specimens with reinforcement ratios of 1.0% and 2.0%. The height of the drop hammer was calculated based on the energy absorption capacity (50%, 75% and 100% energy absorption capacity) from static testing results. For the three beams with a reinforcement ratio of 1.0%, beams were subjected to heights of 355 mm, 533 mm and 710 mm. The three beams with a reinforcement ratio of 2.0% were subjected to heights of 550 mm, 825 mm and 1100 mm. Test parameters investigated included dynamic mid-span deflection, dynamic bending resistance, dynamic strain in GFRP reinforcement bars, failure mode and crack patterns. The GFRP RC beams were labelled according to the series, nominal concrete strength, longitudinal reinforcement type, reinforcement ratio and type of loading. The arrangement is in the form of A–B–C–D where A is the nominal concrete strength (80 or 120 MPa), B is the GFRP reinforcement bar type (#2S, #3HM or #4HM), C is the GFRP reinforcement ratio and D is for the type of loading, static (S) or impact loading (I). For GFRP RC beams under impact loading, the subscript I represents the height of the drop hammer in metres. For example, GFRP RC beam 80–#3HM–1.0–S was designed with concrete compressive strength of 80 MPa with #3HM GFRP reinforcement bars, a longitudinal GFRP reinforcement ratio of 1.0% and tested under static loading. For GFRP RC beam 120–#4HM–2.0–I1,1, nominal concrete compressive strength was 120 MPa, with #4HM GFRP reinforcement bars, a reinforcement ratio of 2.0% and subjected to a 1.1 m height under impact loading.

EXPERIMENTAL SETUP
Static Testing

The beams were simply supported, with a pin support at one end and a roller support at the other end as shown in Figure 1. The simply supported conditions allowed for the GFRP RC beams to deflect under static loading. A 600 kN hydraulic actuator anchored to an independent steel frame was used to apply a monotonic increasing load on a steel circular plate positioned at the mid-span. The hydraulic actuator was also used to measure mid-span deflection. GFRP RC beams were tested under displacement controlled loading at a rate of 1 mm/min until failure. Two electrical resistance strain gauges were attached at the top on each side of the GFRP RC beams, directly underneath the position of the load cell for measurement of concrete strain. One strain gauge was attached to each of tensile GFRP reinforcement bars, at the centre for measurement of tensile strain. All data including load, mid-span deflection and strain were recorded using the high speed data acquisition system.

Figure 1 Experimental Set-up for GFRP RC Beams under Static Loading
Impact Testing

Six GFRP RC beams were subjected to a 580 kg high capacity free falling drop weight apparatus used to apply impact load as shown in Figure 2. The setup procedure involved fixing two steel blocks to the floor to allow for the GFRP RC beams to have a clear span of 2000 mm with 200 mm overhang on each side. All impact GFRP RC beams were simply supported and positioned on a steel pin and steel roller. For the prevention of rebound during impact, steel frame rollers were connected to the steel blocks, which allowed the GFRP RC beams to roll during impact. The drop hammer was mechanically lifted to the required drop height using an automotive control system and released using an electronic quick release system. Dynamic mid-span deflections were determined by image processing technique using high-speed video camera recordings. Dynamic concrete strain was not measured due to the extensive damage in the impact area caused by the drop hammer. However, dynamic tensile strain was measured from the strain gauges located in the middle of each GFRP tensile reinforcement bar. This allowed for an average reading of dynamic tensile strain at the mid-span to be obtained. The recording rate of the high speed camera was 1000 frames per second.

Figure 2 (a) Experimental Set-up for GFRP RC Beams under Impact Loading; (b) Data acquisition system and high speed camera used in impact tests.

EXPERIMENTAL RESULTS AND DISCUSSION

Static Testing

GFRP RC beams under static loading were designed to have two distinct failure modes: GFRP reinforcement rupture and concrete crushing. During testing, the beams designed as under-reinforced displayed vertical flexural cracking, initially forming around the mid-span. Flexural cracks began forming at around 3 kN. At higher loading levels, new vertical cracks began propagating closer towards the supports. Already formed cracks around the mid-span continued to propagate further, vertically. The under-reinforced GFRP RC beams failed due to rupture of the GFRP reinforcement bars as shown in Figure 3, whereas the GFRP RC beams defined as over-reinforced failed by concrete crushing as shown in Figure 4. However, the over-reinforced GFRP RC beams showed signs of continually sustaining load, indicating signs of reserve capacity or an amount of pseudo “ductility”. At higher loading stages, concrete cover continued to crush prior to total failure. At total failure, the over-reinforced GFRP RC beams failed by rupture of the GFRP reinforcement bars. Figure 5 shows the experimental load-deflection graphs for the GFRP RC beams under static loading. All GFRP RC beams displayed a bi-linear relationship. Initially, prior to cracking, beams had high bending stiffness. Once cracking occurred, bending stiffness reduced, especially for the GFRP RC beams with the lowest amount of reinforcement. Concrete strength was shown to be more influential for GFRP RC beams with tensile longitudinal reinforcement ratios of 1.0% and 2.0% in increasing the load carrying capacity, due to the failure being governed by the strength of the concrete (crushing of concrete cover). For reinforcement ratios of 1.0% and 2.0% load increased by 27% (from 33 kN to 41.8 kN) and 13% (from 46.1 kN to 52.2 kN), respectively by increasing concrete from 95 MPa to 116 MPa. At higher reinforcement ratios, higher concrete strength (UHSC) did not show to improve post-cracking bending stiffness. Table 1 reports the experimental load carrying capacity and mid-span deflection of the GFRP RC beams under static loading.
Figure 3 GFRP Reinforcement Rupture of Under-Reinforced GFRP RC Beam

Figure 4 Concrete Crushing of Over-Reinforced GFRP RC Beam

Figure 5 Load-Deflection of GFRP RC Beams under Static Loading
Table 1 Experimental Results for GFRP RC Beams under Static Loading

<table>
<thead>
<tr>
<th>GFRP RC Beam (Failure Mode)</th>
<th>Experimental Load (kN)</th>
<th>Mid-Span Deflection (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>80-#2-0.5 (GFRP Rupture)</td>
<td>15.0</td>
<td>81.8</td>
</tr>
<tr>
<td>80-#3-1.0 (Concrete Crushing)</td>
<td>33.0</td>
<td>62.6</td>
</tr>
<tr>
<td>80-#4-2.0 (Concrete Crushing)</td>
<td>46.1</td>
<td>58.3</td>
</tr>
<tr>
<td>120-#2-0.5 (GFRP Rupture)</td>
<td>16.2</td>
<td>77.5</td>
</tr>
<tr>
<td>120-#3-1.0 (Concrete Crushing)</td>
<td>41.8</td>
<td>73.3</td>
</tr>
<tr>
<td>120-#4-2.0 (Concrete Crushing)</td>
<td>52.2</td>
<td>64.3</td>
</tr>
</tbody>
</table>

Impact Testing

Three GFRP RC beams with reinforcement ratios of 1.0% and 2.0% were subjected to various levels of impact energy. For GFRP RC beam 120II-#3HM-1.0-S, static energy absorption capacities were calculated as 2029 J, 3043 J and 4057 J, at 50%, 75% and 100%, respectively. Hence, three impact heights were calculated as 355 mm, 533 mm and 710 mm, respectively. For GFRP RC beam 120II-#4HM-2.0-S, static energy absorption capacities were calculated as 3189 J, 4783 J and 6377 J, at 50%, 75% and 100%, respectively. Hence, the impact heights were calculated as 550 mm, 825 mm and 1100 mm, respectively. Overall, the experimental failure mode and general behaviour including crack patterns was relatively similar for all six GFRP RC beams subjected to various impact heights. The experimental failure mode, defined as a “dynamic punching failure” was given, which can be defined as the GFRP RC beams being subjected to a moving punch (from the drop hammer) in the mid-span. This resulted in localised concrete crushing on the top surface with the majority of damage (crack propagation) occurring in the impact area.

For the GFRP RC beams with a reinforcement ratio of 1.0%, GFRP RC beam 120-#3HM-1.0-I-0.355 experienced a dynamic punching failure response, with minor crushing of the concrete cover on the top surface, at impact point. During impact, cracks, predominately observed as a combination flexure, flexure-shear and minor shear cracks propagated from the tensile region throughout the height of the GFRP RC beam. Majority of these cracks were observed to be localised around the impact zone, with a few flexure-shear cracks closer towards the supports. No permanent deformation was observed when subjected to an impact energy of 2029 J. GFRP RC beam 120-#3HM-1.0-I-0.533 showed signs of further additional concrete cover crushing, with the exposure of the compressive GFRP reinforcement bars. Crushing of cover was not symmetric under impact, with localisation to one side of the impact point. Under an impact height of 533 mm, a small amount of rupture of the bottom concrete cover occurred, also exposing the GFRP tensile reinforcement bars around the impact zone. This caused a few cracks around the midspan to significantly widen. Cracks were predominately flexure cracks throughout the span of the GFRP RC beam, with the inclusion of a few flexure-shear and minor inclined shear cracks present. GFRP RC beam 120-#3HM-1.0-I-0.710 showed extreme localised concrete cover crushing and rupture of the tensile concrete cover occurred, causing the concrete to spall off as shown in Figure 6. The spalling off of the concrete was shown to be more symmetrical under the impact point, causing exposure of the compressive and tensile GFRP reinforcement bars. Again, a predominant flexural crack pattern was observed around the impact zone, with a very few signs of flexure-shear cracks and minor inclined shear cracking. This GFRP RC beam showed the least number of cracks during impact. By close inspection, some signs of splitting of fibres from GFRP tensile reinforcement bars were observed.

Figure 6 Dynamic Punching Failure and Crack Propagation of GFRP RC Beam 120II-#3HM-1.0-I-0.710
CONCLUSIONS

An experimental study of twelve simply supported GFRP RC beams subjected to static loading and impact loading has been conducted, highlighting the effectiveness of HSC and UHSC. Observations from the experimental testing have led to the following conclusions being made:

1) Failure mode of GFRP RC beams under static loading can be determined using sectional analysis used for beams reinforced with steel reinforcement. It was found that the ratio of the reinforcement ratio to the balanced reinforcement ratio held true for the governing failure. For the GFRP RC beams with more than balanced reinforcement, failure was shown to be caused by crushing of concrete cover. For the GFRP RC beams with less than balanced reinforcement ratio, failure was shown to be caused by GFRP reinforcement rupture.

2) Load-deflection behaviour of GFRP RC beams under static loading showed a bi-linear response. The first section represented an uncracked section, followed by a crack section, resulting in a reduction in bending stiffness. The GFRP RC beams with more reinforcement than balanced reinforcement displayed signs of pseudo “ductility”, where the beams were able to resist load before total collapse. As opposed to the under-reinforced GFRP RC beams which failed suddenly by rupture of GFRP reinforcement, resulting in no reserve capacity.

3) Effect of HSC and UHSC on the GFRP RC beams under static loading were shown to be more influence load carrying capacity, deflection and post-cracking bending stiffness. For the GFRP RC beams with a reinforcement ratio of 0.5%, increasing the concrete strength from 95 MPa to 116 MPa, load increased by 8% (from 15 kN to 16.2 kN). The reason for this is because these GFRP RC beams are designed as under-reinforced and thus their failure is governed by the tensile strength of the GFRP reinforcement bars. For GFRP RC beams with a reinforcement ratio of 1.0% and 2.0%, load increased by 27% (from 33 kN to 41.8 kN) and 13% (from 46.1 kN to 52.2 kN), respectively by increasing concrete from 95 MPa to 116 MPa. However, increasing concrete strength showed to increase mid-span deflection for a reinforcement ratio of 1.0% and 2.0%/m by 17% and 10%, respectively. In terms of post-cracking bending stiffness, for a reinforcement ratio of 1.0%, stiffness increased 10% for a change in concrete strength. However, for a reinforcement ratio of 2.0%, a reduction in 0.07% in post-cracking bending stiffness was observed. At higher reinforcement ratios, higher concrete strength doesn’t seem to improve post-cracking bending stiffness.

4) Under impact loading, regardless of the shear capacity of the GFRP RC beams, the GFRP RC beams displayed a dynamic punching shear failure response. Minor shear cracking around the impact area with crushing of concrete cover was observed. However, the GFRP RC beams under static loading were shown to failure in a flexural response. Thus, the shear behaviour of flexure-critical GFRP RC beams must be considered when designing structures subjected to impact loads.

ACKNOWLEDGMENTS

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REFERENCES


ABSTRACT

The bond between the FRP and concrete plays a critical role in a strengthening system of concrete structures with externally bonded FRP. Modelling of concrete damage and fracture is of crucial importance for any reasonable prediction of the bond behaviour. The dynamic increase factor (DIF) in the strength of concrete-like materials is a subject of extensive investigation and debate for many years. It now tends to be generally accepted that the compressive DIF as observed from standard sample tests is mainly attributable to the dynamic structural effect, whereas for concrete under tension the DIF is deemed to be governed by different mechanisms, probably more from the material at the micro-fracture level, which needs to be modelled as a material property. This paper presents a review of the existing empirical research on the tensile DIF based on the Split Hopkinson pressure bar (SHPB) tests. A numerical investigation is also proposed, with a particular focus on how the DIF, irrespective of its cause, should be included in an appropriate manner in the FE modelling with a local concrete model. The inevitable mesh-dependency issue due to numerical localization and its implications on rate effects in meso-scale modelling of FRP-concrete bond behaviour are also discussed in detail.

KEYWORDS

Dynamic increasing factor (DIF), finite element analysis (FEA), local model, Split Hopkinson pressure bar (SHPB) test, FRP-concrete bond.

INTRODUCTION

Fibre reinforced polymer (FRP) composites have been used to retrofit concrete structures against dynamic loading such as impact (Boyd et al. 2008, Bhatti et al. 2011), blast (Hefferman et al. 2011; Wu et al. 2009; Buchan & Chen 2007; Crawford et al. 1997) and earthquake (Niroomandi et al. 2010; Pan-telides & Gergely 2007; Teng et al. 2007). These studies have confirmed that FRP retrofitting is effective in increasing the structural resistance against these dynamic loading and it has also been observed that debonding on the FRP-concrete interface is one of the predominant failure modes under dynamic loadings. Most of early studies were either experimental (Tarapada and Debrata 2006) or macroscale numerical simulations focused on the global structural behaviour (e.g. Crawford et al. 2001), with limited analytical investigation (De Lorenzis and La Tegola 2005). Little attention has been paid to the FRP-concrete interfacial bond behaviour under dynamic loading.

As many debonding failures occur in the concrete adjacent to the FRP, the modelling of concrete damage and cracking is of crucial importance for any reasonable prediction of the bond behaviour. For concrete structures subjected to rapid loading such as impact and blast, high strain rates in the range of $1 \times 10^3$ are typical. At such high strain rates, the apparent strength of concrete can increase significantly compared with the quasi-static strength. Experimental observations (e.g. Malvar and Crawford 1998) have shown that the dynamic increase factor (DIF), i.e. the ratio of the dynamic to static strength, is a function of the strain rate.

This paper discusses the appropriate incorporation of the DIF in a general finite element framework and its application in modelling FRP-concrete bond behaviour with a local concrete material description. The pertinent issues with regard to strain localisation and the consequent mesh dependence, especially the associated “strain rate localisation” and subsequent need of using a mesh-objective local DIF input versus strain rate relationship, are investigated.
DIF AND ITS NUMERICAL APPLICATION
DIF of concrete materials

The key issue in the debate of DIF is whether the DIF should be treated as an inherent material property, despite that in common practice, e.g. in the widely-used CEB-FIP (1993) model code, the DIF has been introduced as a material property. It tends to be generally accepted that under “uniaxial” compression, the DIF is rather a dynamic structural effect than a material property. Recent studies have demonstrated that the inertia-induced radial confinement makes a large contribution to the dynamic compressive strength enhancement (e.g. Li and Meng 2003; Zhang et al. 2009; Lu et al. 2010). When a high compression stress pulse is imposed to the specimen, the specimen tends to expand in the hoop direction, resulting in a radial inertia force which is equivalent to a confining stress, and subsequently increasing the axial strength of the concrete. As such, it is argued that the concrete DIF in compression should not be imposed at the material constitutive model level, i.e., it should be disabled or simply set as unity for material models that incorporate DIF for compression in a refined finite element analysis. On the other hand, when concrete is under tension, the radial inertia force would change direction (to become radial expanding stresses), so the effect of the lateral (inertia) stress on the axial tensile strength is very different from that of confining stress on the axial compression. Concrete tensile failure is also usually much more localized than that under compression. Furthermore, experimental observations have indicated that the DIF for tension can be considerably larger than that for compression at a comparable strain rate. For example, for a concrete with a static compressive cylinder strength $f'_c = 30$ MPa, when the strain rate is 100$s^{-1}$, the apparent DIF is 8.0 and 2.3 for tension and compression respectively. Such a difference is hardly explicable by the effect of inertia force theory. Numerical studies using the discrete element modelling (e.g. Hentz et al. 2004) or the finite element (FE) modelling (e.g. Lu and Li 2011) of dynamic tensile tests (direct tension, splitting or SHPB spalling), suggested that the DIF of the tensile strength observed in dynamic tensile tests could be a genuine material effect. Consequently, it is deemed to be rational to include the DIF for the dynamic tensile strength in the material description in an analysis where tension plays a dominant role.

The CEB-FIP (1993) recommended DIF curve for compression and Malvar and Crawford’s (1998) tension DIF formula have been commonly adopted for modelling under high-rate loading conditions. Note that in the experimental determination of the apparent dynamic strength using Split Hopkinson pressure bar (SHPB) tests, the strain rate and the global DIF are calculated according to the stress wave time histories measured from the strain gauges attached on the incident and the transmitter bars (e.g. Li and Meng 2003; Tedesco et al. 1991; Tedesco et al. 1989; Ross et al. 1990; Ross 1989). This means that the results are representative of the bulk or macroscopic behaviour of the specimen. When such an “apparent” global DIF is considered in a FE model involving a local concrete model as a material parameter, issues pertinent to the localisation naturally arise, especially when the element size is much smaller than the general crack band width of concrete, which is approximately equal to three times the maximum aggregate sizes (Bazant and Oh 1983).

**DIF mesh correction**

To illustrate the issue related to the localisation of the strain rate in a FE model, a simplified direct tension test was simulated. The DIF curve in Malvar and Crawford (1998) was used directly in the model, whereas different mesh sizes were employed and the results were compared. The specimen used in this analysis is a cylinder with a length of 50.8mm and a diameter of 50.8mm, as typically used in standard dynamic tests (e.g. Tedesco et al. 1991). For illustrative purpose, a simplified 2D plane stress model was employed. To control the occurrence of fracture, a 2x2mm notch was created in the middle of the specimen on both sides. Three different meshes were chosen for the investigation, with element size equal to 2mm, 1mm, and 0.5mm, respectively.

To rectify the mesh-dependent strain rate localisation effect on the DIF, it would be rational to firstly establish a reference characteristic width within which crack softening is deemed to take place in the numerical model. Such a characteristic width may be regarded as a baseline for which empirical tensile DIF formulas in cracking would apply. Similar to Malvar et al.’s (2000) suggestion, herein a characteristic width of 25.4mm (one inch) is adopted, which is roughly three times the size of the 9.5mm (3/8-inch) aggregates that are commonly used in concrete samples. For mesh size differing from (especially smaller than) 25.4mm, the computed strain rate due to localisation should be corrected by the mesh size normalised with respect to the characteristic width before the empirical DIF formula is applied. This is equivalent to introducing a correction to the local DIF formula as an input to the model:

$$DIF = f_l / f_{tt} = (\frac{\dot{\varepsilon}}{\dot{\varepsilon}_c})^\gamma \quad \text{for} \quad \dot{\varepsilon} \leq \frac{\dot{\varepsilon}_{c0}}{x} s^{-1}$$

(1a)
where $X$ is the current element size in mm; $L_{x0}$ represents the characteristic width of the current concrete model in mm, herein 25.4 mm. Three loading rates were then simulated, namely 5, 50 and 200 mm/s ($\dot{\varepsilon} = 0.1, 1.0$ and 4.0 s$^{-1}$), respectively. The results are compared with Ross’s (1989) test data together with the global DIF curve proposed by Malvar et al. (2000) and they. The FE predictions for the direct tension test with the proposed mesh-corrected local DIF in Equations (1a) and (1b) are in good agreement with the empirical global DIF data, especially for strain rates up to the order of 1 s$^{-1}$. Further simulation for higher strain rates, however, did not produce as such “good” results. The DIF from FE simulations with local DIF curve expressed in Equations (1a) and (1b) for 4 s$^{-1}$ strain rate is lower than the value based on the modified CEB curve and experimental results. This is explicable by the fact that the simple correction in Equations (1a) and (1b) assumes the localisation of strain (and strain rate) within a single element, so the validity is limited to cases where failure does localise in a single element width. This is usually valid under quasi-static and relatively low strain rate dynamic tension. When the loading rate is higher, however, damage tends to distribute in a wider area due to strong transient dynamic effects. Such a phenomenon renders the localisation correction more complicated and warrants further investigation, as follows.

Mesh and rate dependency of DIF

As the simplified direct tension model cannot properly represent the real concrete behaviour under higher strain rates, the split-Hopkinson pressure bar (SHPB) test for direct tension of concrete was modelled with the inclusion of the pressure bars and the concrete specimen. The SHPB setup for direct tension as developed by Ross (1989) was adopted in this study. Two strain gauges are located at the middle of the incident and transmitted bars, so that the stress wave time histories can be measured. During the test, a striker impacts a tup which is attached at the end of what becomes the tensile incident bar. The overall dimension of the tested specimen was 50.8 mm (2 inches) in diameter and 50.8 mm in length. A 3.175 mm square notch was located at the mid-length of the specimen. The pressure bars were made of PH 13-8 MO stainless steel. The concrete being tested had the following properties: Young’s modulus $E_s = 37.93$ GPa; compressive stress $f'_c = 57.7$ MPa; tensile strength $f_t = 4.53$ MPa; density $\rho = 2405$ kg/m$^3$ (Tedesco et al. 1991). For simplicity, this SHPB test is treated as a plane stress problem in the numerical modelling, as was done in some previous studies (e.g. Zhou and Hao 2008; Tedesco et al. 1991), and only half (along the longitudinal symmetry plane) of the SHPB is modelled considering symmetry. A tensile stress pulse is imposed at the end of the incident (right side) bar instead of modelling the striker.

Different tensile impulses can be imposed on the incident face of the pressure bar to achieve different strain rates (e.g. Tedesco et al. 1991). To examine the adequacy of considering the local DIF at the material level, two options were examined with the current concrete model, one without any DIF, and the other with a local DIF which incorporates the simple mesh correction (Eq. 1). The results showed that a further correction is necessary to take into account the tendency of an increasing spread of tension failure in the higher strain regime. Combining with the mesh correction, this provides a more robust model for dealing with the issues in the FE modelling for dynamic tension arising from strain / rate-localisation and its increased complexity due to transient dynamic effect. Equations (2a) to (2d) present a “double-corrected” DIF curve for local concrete model. The strain rate correction of the localisation is achieved by introducing a dynamic characteristic length, such that $L_{x0} = L_{x0}(1+\xi \log \dot{\varepsilon})$ for $\dot{\varepsilon} > \dot{\varepsilon}_c$, where $\xi$ is an empirical coefficient, $\dot{\varepsilon}_c$ is a numerical critical strain rate below which only the mesh correction on the DIF is required.

$$DIF = f_i / f_n = \frac{\dot{\varepsilon}}{\frac{L_{x0}}{\dot{\varepsilon}_t}}^y \text{ for } \dot{\varepsilon} \leq \dot{\varepsilon}_t$$

(2a)

$$DIF = f_i / f_n = \left( \frac{\dot{\varepsilon}}{\frac{L_{x0}(1+\xi \log \dot{\varepsilon})}{\dot{\varepsilon}_t}} \right)^{1/3} \text{ for } \dot{\varepsilon} \leq \dot{\varepsilon}_c$$

(2b)

$$DIF = f_i / f_n = \beta \left( \frac{\dot{\varepsilon}}{\frac{L_{x0}}{\dot{\varepsilon}_t}} \right)^{1/3} \text{ for } \dot{\varepsilon} \leq \dot{\varepsilon}_c$$

(2c)

$$DIF = f_i / f_n = \beta \left( \frac{\dot{\varepsilon}}{\frac{L_{x0}(1+\xi \log \dot{\varepsilon})}{\dot{\varepsilon}_t}} \right)^{1/3} \text{ for } \dot{\varepsilon} \leq \dot{\varepsilon}_c$$

(2d)
\[ \xi = 6.8 \left( \frac{1}{L_{\text{so}}^2} \right) - 1 \quad (2c) \]

\[ \frac{e_0}{L_0} \delta \left( \frac{e_0}{L_0} \right)^{\beta} = \beta \left( \frac{e_0}{L_0} \left( 1 + \frac{\xi}{\varepsilon} \frac{\delta}{\varepsilon} \right) \right)^{1/3} \quad (2d) \]

MODELLING FRP-CONCRETE BOND WITH DIF

The numerical modelling of the impact test

The K&C concrete damage model (Malvar et.al 2000) with the above “double corrected” concrete local DIF was next introduced to perform meso-scale modelling of a series of FRP-to-concrete bond impact test conducted by Chen et al. (2012) where details of the test specimens can be found.

The beam specimen was idealized as a 2D plane stress problem so only a single layer of concrete element in the through thickness direction was modelled using the 3D solid element in LS-DYNA. Only half of the specimen was modelled considering symmetry. A pin-end boundary condition was applied. Both FRP and steel reinforcement bar was assumed to be perfectly bonded to the concrete. The impact load (Figure 1) was treated as a uniformly distributed pressure load applied at an \(100 \times 100 \text{mm}^2\) impact zone. Three element sizes, 10mm, 5mm, 2.5mm, were used for mesh convergence analysis (Figure 2), and the finally adopted mesh consists of 2.5mm square elements. The concrete compressive cylinder strength \(f_c\) was 25 MPa.

![Figure 1 Impact pressure load time history](image1)

![Figure 2 SG1 readings for test IT2](image2)

It can be seen from Figure 2 that the FE prediction at SG1 is in very good agreement with the test result. However, the other strains outputs do not match with the test results well: The closer the strain gauge location to the mid span where debonding starts, the better the FE strain prediction is in agreement with the test as shown in Table 1. Note that the strain gauge at SG3 was faulty in the test.

<table>
<thead>
<tr>
<th>Test (IT2)</th>
<th>Pressure load_10mm mesh</th>
<th>Pressure load_5mm mesh</th>
<th>Pressure load_2.5mm mesh</th>
</tr>
</thead>
<tbody>
<tr>
<td>SG1</td>
<td>3000</td>
<td>2918</td>
<td>N/A</td>
</tr>
<tr>
<td>SG2</td>
<td>2000</td>
<td>1212</td>
<td>726</td>
</tr>
<tr>
<td>SG3</td>
<td>2500</td>
<td>2147</td>
<td>384</td>
</tr>
</tbody>
</table>

The poor match for strain measurements except that at SG1 is mostly due to the difference in the failure mode. Total debonding occurred at the test, but only at less than 50mm from the middle gap in the FE simulation. Among various possible causes, the most plausible could be: 1) the FRP-to-concrete bond was in a poorer state in the test so that the FE model over estimates the strength, and 2) the impact load is under estimated. These two factors are investigated below.

Discussion of test results

A numerical attempt was done with an impact load 20% higher than the measured from test IT2. The predicted damage pattern is shown in Figure 3. Clearly the debonded length in this case is longer than the original one. The strain outputs are compared in Figure 4.
From Figure 4, the FE predicted peak strain at SG1 increases slightly due to the increase of the impact load. However, the predicted SG2 strain increases significantly and give a good agreement with the test value, because the debonding length for the case with 120% impact load is over 50mm starting from the middle span where SG2 is located at 50mm away from the mid span. For the case when 100% measured impact load is applied, the SG2 output is much lower than the test results as the debonding has not been developed so far. The strain outputs of SG4 and SG5 also indicate that more damage occurs at the bond, the closer the strain outputs agree with the test results. The unmatched SG4 and SG5 FE strain outputs to the test results are still due to the debonding failure does not happen there. Accordingly, it is clear that there are two main possible causes: 1) The inaccuracy of the load measurement. As presented in Figure 4, with an additional 20% impact load, the predicted SG 1 value is only slightly increased whereas the debonded length is increased and SG2 output is significantly increased. There was just one striker available with the load cell fixed to it at the time of test. The measurement range of the load cell in the test was up to 2000 kN (Chen et al. 2012). Due to the low impact heights used in the test; the peak load measured was only around 100 kN for test IT4 and 35 kN for test IT2. 2) The method of surface treatment. The surface treatment might not offer sufficient bond strength in the test. In the test, total debonding occurred with less than 1mm mortar attached to the FRP. However, in the FE total debonding failure does not occur. In the numerical model the debonding depth of the concrete adjacent to FRP was thicker due to the ideal bond assumption.

CONCLUSIONS

This paper has presented a study which shows that the commonly adopted empirical global DIF model cannot be applied directly in the FE analysis with a local concrete material model due to strain and strain rate localisation. A new “doubly corrected” local DIF model has been proposed considering both element size and strain rate effects. A preliminary study on the effect of dynamic loading rate on the behaviour of FRP-to-concrete bonded joint has been presented. By deploying the K&C concrete damage model and the proposed local DIF model, the FE predictions have been shown to be in close agreement with the test results.

ACKNOWLEDGMENTS

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State Education Ministry (Project No. 2013-1792) and a scholarship provided to the first author by EPSRC (UK) and Royal Dutch Shell plc through a Dorothy Hodgkin Postgraduate Award.

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TEMPERATURE EFFECT ON THE DYNAMIC BOND BEHAVIOR OF CFRP/STEEL SINGLE-LAP SHEAR JOINTS

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ABSTRACT
In this work, CFRP plates were made using vacuum assisted resin transfer molding (VARTM) technique. CFRP/Steel single-lap shear joints were manufactured by bonding steel and CFRP together with epoxy resin. Specimens were tested under dynamic tensile loading (0.625 m/s) on a servo-hydraulic high rate testing system under four temperatures (-25, 0, 50, 100 °C). The experimental results showed that the mechanical properties of joints were sensitive to temperature. In a certain range, both the bond strength and interfacial fracture energy of specimens increased from -25 to 50 °C. When the temperature exceeded the glass transition temperature of the adhesive, bond strength and interfacial fracture energy decreased significantly. The failure modes of specimens were different at varying temperatures. At the low temperature (-25 °C and 0 °C), the failure mode of joints were adhesive/steel interface debonding failure. At the temperature of 50 °C, the failure mode turned to the mixed failure. When the temperature reached 100 °C, the failure mode changed to adhesive/CFRP interface debonding failure. Moreover, the strain field distribution of FRP in overlapping area at different temperatures was analyzed using a digital image correlation (DIC) method. The results showed that the strains of FRP at the edge of overlapping area were larger than those in the center, and final failure was initiated at the edge of the specimen.

KEYWORDS
Single lap shear joints; Temperature effect; Bond strength; Failure mode

INTRODUCTION
FRP materials are especially useful in a wide-range of applications such as strengthening and rehabilitation of structures (Gholami et al. 2013; Zhao and Zhang 2007). Their high strength to weight ratio, attractive energy dissipating, corrosion resistance, and lower manufacturing costs enable them to be very efficient when compared with conventional metals for use in building reinforcement. The most commonly used FRP for the strengthening of steel structures is carbon fibre-reinforced polymer (CFRP), owing to its high strength and high modulus. One of the relevant issues is the bond mechanical performance between CFRP and steel (Schnerch et al. 2006; Korayem et al. 2015), as it significantly influences the mechanical properties of the long service performance. Extensive researches (Wu et al. 2012; Liu et al. 2010) had already been conducted on CFRP/steel double strap joints for the understanding of the mechanics and failure mechanisms. Liu et al. (2010) studied the effect of fatigue loading on bond strength and failure modes, and reported that fatigue loads have little effect on the bond strength and failure modes of high modulus CFRP-sheet bonded steel plates. Wu et al. (2012) performed the bond tests of ultra-high modulus CFRP laminates and steel plates, and described bond strength, effective bond length, failure pattern, CFRP strain distribution and adhesive shear stress distribution along the overlapping areas. Haghani (2010) dealt with analyses of adhesive joints used to bond CFRP laminates to steel substrates using a numerical and experimental approach. Different aspects of joint behaviour, such as strain distributions along bond line and through the thickness of the adhesive layer and failure mechanisms are discussed. The aforementioned researches were conducted at ambient temperature. However, information on CFRP/steel adhesively joints subjected to varying temperatures remains limited (Nguyen et al. 2011), although such high or low temperature is realistic in different regions.

In this paper, a series of CFRP/steel single-lap shear joints were examined in tension in the range of -25 to 100 °C at the same loading rate of 0.625 m/s. The primary objective of the experimental research is to investigate the effect of temperature on the mechanical properties of joints under dynamic tensile loadings. In the next section the
EXPERIMENTAL PROGRAM

High speed tensile test methodology

The high speed tensile tests were conducted using a servo-hydraulic testing machine with a load capacity of 25 kN operating under open-loop mode. The speed of the actuator was controlled by the servo-valve of hydraulic supply. By manually turning the servo-valve, the rate of flow of hydraulic fluid can be controlled, resulting in different stroke speeds. The setup of the dynamic tensile testing is presented in Figure 1. An environmental chamber, temperature controller and liquid nitrogen were employed for testing under varying temperatures. A Phantom v7.3 high speed digital camera with sampling rate of 10,000 fps captured the deformation, cracking, and failure behaviour of the full size of the specimen in between the grips. More details of the testing system can be found elsewhere (Zhu et al. 2010).

Figure 1 Experimental facility (MTS servo-hydraulic high-rate testing machine)

Specimen preparation

The carbon unidirectional fiber fabric made by Nanjing Hitech Composites Co., Ltd (Nanjing, China) was adopted in this study. Hot rolled steel plates (Q235, a nominal yield stress of 235 MPa, China standard, GB/T 700) were chosen to prepare CFRP/steel single-lap shear joints. Modified epoxy resin provided by Hunan Good Bond Construction Technic Development Co., Ltd (Changsha, China) was utilized as matrix to manufacture CFRP laminates by using vacuum-assisted resin transfer molding (VARTM) technology. Details about physical and mechanical properties of these materials are listed in Table 1.

<table>
<thead>
<tr>
<th>Material</th>
<th>Young's modulus (GPa)</th>
<th>Elongation (%)</th>
<th>Yield strength (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>Coefficient of thermal expansion (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sikadur 330</td>
<td>4.6</td>
<td>1.0</td>
<td>-</td>
<td>38.1</td>
<td>4.5 x 10^{-5}</td>
</tr>
<tr>
<td>Steel plate (Q235)</td>
<td>199</td>
<td>35.6</td>
<td>235</td>
<td>441</td>
<td>1.3 x 10^{-5}</td>
</tr>
<tr>
<td>Carbon fiber fabric (single layer)</td>
<td>235</td>
<td>1.60</td>
<td>-</td>
<td>4200</td>
<td>-</td>
</tr>
</tbody>
</table>

The joints were formed using a wet lay-up method. The steel plates and CFRP laminates were cut to the required dimension of 25 mm × 80 mm × 4 mm (width × length × thickness) and 25 mm × 80 mm × 0.95 mm (width ×
length \times thickness), prior to the fabrication of single-lap CFRP/steel shear joints. The surface of steel plates were sandblasted and cleaned with acetone to remove grease, oil and rust. Steel plates and CFRP sheets were bonded together with an overlapping length of 25 mm by epoxy resin Sikadur 330, which is widely used in the FRP/concrete system. Two end spacers (CFRP and steel) of 25 mm long, 25 mm wide were glued on each side of specimens for loading applied and easy alignment, as shown in Figure 2. Great attention was paid to the thickness of the adhesive layer, which influences the mechanical properties of the bonding (Arenas et al. 2010). The final steel/CFRP single lap shear joints were fabricated as shown in Figure 6.

![Figure 2 Schematic view of steel/CFRP single-lap joints (not to scale): (a) front view; (b) side view](image)

**RESULTS AND DISCUSSIONS**

**Temperature effect**

The results of the experimental work are presented and discussed in this section. Dynamic tensile tests were undertaken with four temperatures of -25, 0, 50, 100 °C. The bond stress is given by:

![Figure 3 Bond Stress vs. Displacement curves at different temperatures: (a) -25 °C, (b) 0 °C, (c) 50 °C, (d) 100 °C](image)
\[ \tau = \frac{F}{b \times l} \]  

(1)

Where \( \tau \) is the bond stress, in MPa, \( F \) is the tensile load, in N, \( b \) is the joint width and \( l \) is the joint overlap length, in mm. Interfacial fracture energy is evaluated using the area under the bond stress-displacement curve, as defined in Eq. (2).

\[ G_f = \int_{0}^{s_f} \tau ds \]  

(2)

Where \( G_f \) is the interfacial fracture energy, in MPa-mm, \( s \) is the displacement and \( s_f \) is the maximum displacement, in mm. Shear stiffness is defined as the slope of the curve in the elastic region.

Figure 3 illustrates the experimental bond stress-displacement responses of specimens at the same loading rate of 0.625 m/s. The responses of individual specimens are quite different. It is obvious that these curves show different degrees of discreteness, especially at the temperature of 50 °C, the ascending branch and descending branch of each curve show large difference. In fact, dynamic tests always show relatively larger scatter compared to quasi-static tests. The reason for the difference at 50 °C is that when the temperature is close to the glass transition temperature of epoxy resin, the adhesive becomes unstable, therefore the dynamic tensile shear mechanical properties of the specimens are seriously affected. While the temperature reaches 100 °C (already higher than \( T_g \)), the adhesive has turned into a soft elastic plastic body, thus the bond tension between CFRP and steel plates is reduced, causing the decrease of bond strength.

Figure 4 shows the experimental observations of the influence of temperatures on the dynamic mechanical properties of specimens. It indicates the temperature-sensitivity of the CFRP/steel adhesively bonded joints. Overall, both bond strength and interfacial fracture energy of joints increase from -25 to 50 °C and decrease from 50 to 100 °C. The bond strength increases from 4.50 ± 1.23 MPa at a subzero temperature of -25 °C to 6.59 ± 1.77 and 7.54 ± 1.91 MPa at the temperature of 0 °C and 50 °C. Interfacial fracture energy increases in the range of -25 °C to 50 °C. However, when the temperature increases from 50 °C to 100 °C, bond strength decreases about 31% from 7.54 ± 1.91 MPa to 5.22 ± 3.02 MPa, interfacial fracture energy decreases from 5.34 MPa-mm to 4.88 MPa-mm, which can be attribute to the softening of the adhesive when the \( T_g \) is reached. Besides, the trend of shear stiffness as a function of temperature decreases almost linearly. Specifically, shear stiffness decreases as much as 68% from 7.78 MPa/mm to 2.52 MPa/mm with temperature increasing from -25 to 100 °C.

Figure 4 Temperature effect on the mechanical properties of single-lap shear joints: (a) bond strength, (b) interfacial fracture energy, (c) shear stiffness
Failure mode

Figure 5 compares the failure modes of the specimens at different temperatures under high speed tensile loads, which clearly shows a remarkable difference in the overlapping area. It is obvious that the actual failure is different from the expected in-adhesive failure, i.e., the fracture is partially in-adhesive and partially interfacial in nature. Three failure modes are observed in the test specimens, including adhesive/CFRP interface debonding, adhesive/steel interface debonding and mixed failure mode. The mixed failure mode is defined as some adhesive remaining on CFRP and steel. At low temperature (-25 and 0 °C), the failure modes are mainly adhesive/steel interface debonding, as shown in Figure 5a and Figure 5b. However, when the temperature increases to 50 °C, the inside adhesive between CFRP and steel begins to disintegrate, part of adhesives remains on the steel and CFRP, as shown in Figure 5c. When the temperature exceeds the $T_g$ of the adhesive, bond strength decreases rapidly and failure mode changes to adhesive/CFRP interface debonding.

Image analysis (DIC method)

As a full-field approach, image analysis techniques are widely used in different aspects of fields such as civil engineering (Corr et al. 2007), mechanical engineering (Helfrick et al. 2011), material science (Catalanotti et al. 2010) and biomedical engineering (Thompson et al. 2007). The use of DIC to obtain overlapping area (FRP) strains in joints is relatively new. By performing digital image processing on the images of the test specimen before and after applying loads, the entire displacement field can be constructed.

Figure 5 Fracture morphologies under different temperatures: (a) -25, (b) 0, (c) 50, (d) 100 °C

Figure 6 Painted specimen with speckle pattern
Figure 6 shows the random speckle pattern and area of interest (AOI) selected in the current study. A commercial software Vic-2D 2009 developed by Correlated Solutions, Inc. was used to perform image analysis. As shown in Figure 7, a relative uniform distribution of strains is obtained at the beginning of the test. As the load increases, normal strain starts to localize at the edge of the AOI. The localization of normal strain indicates the concentration of stress along the edge of the adhesive while the far fields are still well bonded. Then the localization zone continues to grow and propagate towards the inner areas. The development of non-uniform strain fields implies the fact that the damage of the adhesive joint may initiate at the edges and spread out to the far fields. It is worthy to mention that the pattern of strain fields seems to be independent on the effect of temperature.

Figure 7 CFRP strain-field in the overlap area under different temperatures: (a) -25, (b) 0, (c) 50, (d) 100 °C

CONCLUSIONS

This paper presents the dynamic tensile properties and failure morphology of CFRP/steel single lap joints at four temperatures by using a servo-hydraulic high-rate testing machine with an environmental chamber. The results and discussions presented in the paper allow the following conclusions to be made:

(1) Bond strength increases 68% from -25 °C to 50 °C, and interfacial fracture energy increases from -25 °C to 50 °C. While the temperature increases from 50 to 100 °C, bond strength decreases about 31% from 7.54 ± 1.91 MPa to 5.22 ± 3.02 MPa, interfacial fracture energy decreases from 5.34 MPa•mm to 4.88 MPa•mm.

(2) Three failure modes are observed in the current work. Most specimens show adhesive/steel interface debonding failure mode. When the temperature increases to 50 °C, the failure mode changes to mixed failure mode. At the temperature of 100 °C, the failure mode turns to adhesive/CFRP interface debonding.

(3) DIC method can better analyze the failure process and displacement/strain field of the specimens. The failure initiates from the upper and low edges along the bond area. Higher strains were observed at the edge of the bond line and lower strains at the center.
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REFERENCES

TRANSVERSE RESISTANCE OF CFRP CABLES UNDER DROP-WEIGHT IMPACT

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ABSTRACT

Bridge cables may be subjected to lateral impacts caused by vehicle collisions when in service. When Carbon-Fiber-Reinforced Polymer/plastic (CFRP) cables are used to replace traditional steel cables, their impact behavior warrants further investigation. However, to date, few investigations have focused on the impact behavior of CFRP cables. In this paper, drop-weight tests were conducted on three scaled CFRP cables. Each specimen was composed of a seven-wire CFRP strand and two reactive powder concrete (RPC)-grouted bond-type anchors at both ends of the specimen. Results showed that the damage patterns include indentation and complete fracture of the CFRP cable. The anchor with a bond-length of 250 mm provided the CFRP strand with a sufficient anchorage under impact. The transverse impact resistance and energy dissipation capacity were 16 kN and 727 J, respectively, for a CFRP cable under a pretension of 40 kN.

KEYWORDS

CFRP cable, impact, drop-weight test, strand, resistance.

INTRODUCTION

Bridge cables may be subjected to lateral impacts caused by vehicle collisions when in service, therefore, when employ carbon-fiber-reinforced polymer/plastic (CFRP) cables to replace traditional steel cables, the impact behavior of CFRP cables warrants further investigation. To date, extensive research has been conducted on the impact behavior of CFRP laminates, but few if any such investigations have focused on CFRP cables. Experiments were carried out to study the macro damage phenomena and few influential factors that affect the impact behavior of CFRP laminates under impact (Cantwell and Morton 1989; Ghelli and Minak 2011; Mitrevski et al. 2006). Regarding the composite components that are used in the field, they are usually under initial stresses before being subjected to impact. Thus, the effect of preload on the impact resistance, stiffness and energy dissipation capacity of CFRP laminates have been investigated using experiments (Sun and Chen 1985; Whittingham et al. 2004; Heimbs et al. 2009; Saghaifi et al. 2014;) and analytical predictions (Khalili et al. 2007). This paper presents drop-weight tests on three scaled CFRP cables to investigate their impact behavior which includes the damage pattern, transverse impact resistance, and energy dissipation capacity.

EXPERIMENTAL PROGRAM

Specimen Preparation

The specimens were designed to have the same slenderness ratio and initial stress level as a suspender in a prototype bridge with a main span of 856 m. Each scaled specimen was composed of a Φ12.5 mm seven-wire CFRP strand and two bond-type anchors grouted by RPC at each end. The specimen had a full length of 2000 mm and a free length of 1500 mm, as shown in Figure 1(a). The anchor at each end of the specimen had a fixed bond-length of 250 mm with external M50×3 thread for imposing the pretension. The CFRP strand was untwisted inside the anchor tube, to achieve well anchoring performance of the anchor, as shown in Figure 1(b).

Figure 1 The specimen: (a) overall geometry; and (b) details of the untwisted CFRP strand
The CFRP strand used in present study was manufactured by Tokyo Rope Company. It had feature and configuration similar to the traditional steel strand, and was composed of seven twisted CFRP wires, as shown in Figure 2. The nominal diameter and effective cross-sectional area of the CFRP strand were measured to be 12.54 mm and 76 mm², respectively. The average tensile breaking load, tensile strength, and the modulus of elasticity of the CFRP strand were measured to be 193 kN, 2539 MPa and 157 GPa, respectively.

Figure 2 1×7Ø12.5 mm CFRP strand used in present study

Test setup and method

Figure 3 illustrates the drop-weight impact test setup. A special fixture was designed to provide each specimen with enough support. Two test pedestals at both ends of the specimen were connected by four Φ48 mm high-strength screws. Moreover, the whole fixture was bolted to the concrete foundation of the laboratory to reduce vibration during the impact.

Prior to an impact test, a pretension of 40 kN was applied to a specimen. Then a drop hammer with a mass of 187 kg was dropped freely onto the mid-span of the specimen from three different heights: 200 mm, 600 mm and 1000 mm. The drop hammer has a semi-cylindrical impact head with a radius of 20 mm. The impact force was measured through a piezoelectric load washer built into the hammer. The transverse deflection and axial deformation of the specimen were measured by three potentiometers. The data acquisition system recorded the impact force and deflection at a sampling rate of 500 kHz.

Figure 3 Drop-weight impact test setup

RESULTS AND DISCUSSIONS

An overview of the impact conditions and main results are summarized in Table 1. The nomenclature indicates the specimen number and the drop heights. For example, specimen code C3-H1000 means the third specimen of the present study was subjected to a drop weight impact from a 1000 mm height.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Eimp (J)</th>
<th>H (mm)</th>
<th>Fmax (kN)</th>
<th>tδ (ms)</th>
<th>dmax (mm)</th>
<th>du (mm)</th>
<th>Edis (J)</th>
<th>damage pattern</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1-H200</td>
<td>367</td>
<td>200</td>
<td>12.67</td>
<td>100</td>
<td>57.31</td>
<td>--</td>
<td>45</td>
<td>Indentation</td>
</tr>
<tr>
<td>C2-H600</td>
<td>1100</td>
<td>600</td>
<td>15.75</td>
<td>51</td>
<td>61.62</td>
<td>61.62</td>
<td>725</td>
<td>Fracture</td>
</tr>
<tr>
<td>C3-H1000</td>
<td>1833</td>
<td>1000</td>
<td>15.96</td>
<td>33</td>
<td>69.33</td>
<td>69.33</td>
<td>728</td>
<td>Fracture</td>
</tr>
</tbody>
</table>

Note: \( E_{imp} \) = the incident impact energy, calculated by \( E_{imp} = mgH \); \( H \) = the drop height; \( F_{max} \) = the maximum impact force; \( t_δ \) = the contact duration for impacting; \( d_{max} \) = the transverse deflection at the \( F_{max} \); \( d_u \) = the transverse deflection when the specimen fractured; \( E_{dis} \) = the dissipated energy.
Damage pattern

Two typical damage patterns observed after the drop-weight tests are shown in Figure 4. The first pattern of indentation occurred under a relatively low incident impact energy, corresponding to specimen C1-H200, the hammer hit the CFRP strand and rebounded, and an indentation was observed afterward on the top surface of the CFRP strand, see Figure 4(a). The second pattern of complete fracture occurred under high incident impact energies, corresponding specimens C2-H600 and C3-H1000, the seven wires of the CFRP strand were successively cut off during the impact process, see Figure 4(b). In addition, the anchor with a bond-length of 250 mm provided the untwisted CFRP strand with sufficient anchorage under the impact, because no obvious anchor slip was observed among all specimens, as shown in Figure 5.

Impact force and energy deflection curves

Figure 6 shows the impact force versus deflection curves for specimens under different incident impact energies. It can be observed that the features of the curves are closely related to the damage patterns. For the indentation-type pattern, specimen C1-H200 exhibited almost overlapping ascending and descending phase of the curve. The impact force and deflection simultaneously increased to their maximum values of 12.7 kN and 57.3 mm, respectively, and eventually returned to zero after the rebound of the hammer. Little reduction is evident in the slope of the ascending and descending phases. Moreover, concerning the pattern of fracture, both specimens C2-H600 and C3-H1000 presented open-type curves. The impact forces for specimens C2 and C3 linearly increased to their maximum values of 15.8 kN and 15.9 kN, at deflections of 61.6 mm and 69.3 mm, respectively. Then a subsequent stepwise decrease in the impact force accompanied by a continuous increase in deflection can be observed from each curve of specimen C2 and C3, reflecting the sequential fracturing of seven individual wires. The maximum values of impact forces initially increased with the increasing incident impact energy and then reached a plateau value as the transverse impact resistance of the CFRP cables was approached, as shown in Figure 7. Consequently, the transverse impact resistance of the CFRP cable under a pretension of 40 kN was determined to be approximately 16 kN.
The energy transferred from the hammer to the specimen in an impact event can be calculated by integrating the impact force-deflection curve, as given in Figure 8. The specimen C1-H200 showed that the energy increased with the deflection at the ascending phase and gradually recovered at the descending phase. It can be observed that approximately 327 J energy was transferred to the specimen C1, then most of the energy was transferred back to the hammer for the rebound, and only 45 J energy was permanently dissipated ($E_{dis}$) as the damage pattern of indentation. Concerning fractured specimens C2-H600 and C3-H1000, despite incident impact energies for these two specimens were 1100 J and 1833 J, just 725 J and 728 J energies transferred to specimens C2 and C3 and fully dissipated by the fracture of the specimens, respectively. The excess energies remained in the hammer after the complete fracture of the specimens. From the relationship between the dissipated energy and incident impact energy, as shown in Figure 9, it can be concluded that the average energy dissipation capacity of a CFRP cable under a pretension of 40 kN is approximately 727 J.

### CONCLUSIONS

This study investigated the impact behavior of the CFRP cables under pretention by drop-weight tests. The conclusions can be drawn as follows:

1. Typical damage patterns of the CFRP cables, including indentation and complete fracture, were observed to be closely related to the incident impact energies.
2. No obvious anchor slip was observed among all specimens, which indicated that the anchor with a bond-length of 250 mm provided the untwisted CFRP strand with proficient anchorage under impact.
3. The transverse impact resistance and energy dissipation capacity were 16 kN and 727 J, respectively, for a CFRP cable under a pretension of 40 kN.

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THE CHARPY IMPACT BEHAVIOR OF CFRP RODS AND STRANDS

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ABSTRACT
An increasing application of Carbon-fiber-reinforced-polymer (CFRP) in civil structures has been witnessed in recent years, however, the impact behavior of CFRP rod and CFRP strand failed to meet the expectation whilst was seldom reported. In present paper, the impact behavior of CFRP rod and strand was investigated by conducting Charpy impact tests on four groups of specimens, which composed of CFRP rod, entire CFRP strand, central and outer individual wire of CFRP strand. All specimens were cut into 100 mm long and impacted by a Charpy impact energy of 300 J. Test results showed that the damage modes include debonding between fibres and matrix of CFRP rod and wire decentralization and fracture of CFRP strand. The average absorbed energies for CFRP rod and CFRP strand are 45.3 J and 32.9 J, respectively; the average absorbed energies for central and outer individual wires of a CFRP strand are both 2.5 J.

KEYWORDS
CFRP rod, strand, Charpy impact, absorbed energy.

INTRODUCTION
Carbon-fiber-reinforced-polymer (CFRP) has recently been widely used in many fields, due to its advantages of light weight, high strength and excellent durability, as well as the convenience of forming into components with complex shapes. However, this kind of material is well accepted to be vulnerable to transverse load, especially under impact event (Reid and Zhou. 2002).

To date, much effort has been spent on the impact behavior of CFRP laminates. Sohn and Hu (1996) researched the delamination mechanisms and energy dissipation of CFRP under impact; Hufenbach et al. (2008) investigated the influence of key parameters, including fiber reinforced type, hybrid materials, fiber type and core fiber orientation, on the impact behavior of CFRP with the Charpy impact test; Ghasemnejad et al. (2010) experimentally investigated the Charpy impact response of hybrid delaminated composite beams and simulated the impact process of composite beams with LS-DYNA. The research on impact behavior of CFRP rod and CFRP strand were seldom reported. Therefore, in the present study, Charpy impact tests were conducted to examine the transverse impact behavior of CFRP rod, entire CFRP strand, central and outer individual wires of the CFRP strand. The energy absorption capacities of these specimens were obtained, and damage modes after failure were observed.

MATERIALS AND SPECIMENS
Two types of CFRP tendon adopted in present research, including the CFRP rod and the CFRP strand, are shown in Fig 1. The CFRP rod has a nominal diameter of 9.8 mm and a deformed surface wrapped in helical rib. The post-processed rib was designed with a width of 6 mm, thickness of 0.15 mm and interval of 10 mm. The cross-sectional area of the core part of the rod is measured to be 71 mm². In addition, the average tensile breaking load and the ultimate tensile strength of the CFRP rod are 187 kN and 2655 MPa, respectively.

CFRP strand, with the same appearance of common used steel strand, was made by twining six spiral outer CFRP wires around one straight central CFRP wire with a fixed spiral pitch. All wires were numbered from 1# to 7#, as shown in Fig 1(b). The nominal diameter and effective sectional area for individual wires of the CFRP strand are 4.16 mm and 10.1 mm², respectively. As for the entire CFRP strand, its nominal diameter and effective sectional area are 12.48 mm and 78 mm², respectively. The average breaking load of the CFRP strand is 195 kN and the ultimate tensile strength is about 2539 MPa. Moreover, epoxy resin is filled in the gapping place among wires in CFRP strand to bond wires together and to improve the integrity of CFRP strand.
In accordance with the test standard (ISO179-1, 2001), four groups of specimens with the same length of 100 mm were prepared for the Charpy impact tests. These specimens composed of CFRP rod, entire CFRP strand, central wire of CFRP strand and outer wire of CFRP strand, and were marked as CR, CS, CS-C and CS-O, respectively. The photographs of typical specimens are shown in Fig 2.

**EXPERIMENTAL PROCEDURE**

The impact tests were conducted with a pendulum Charpy impact system, as seen in Fig 3. The Charpy device is a dynamic three-point bending experiment of an un-notched beam. The experimental setup consists of the specimen, the anvils where the specimen is freely supported, a pendulum with a defined mass attached to a rotating arm pinned at the machine body. The pendulum falls along a circular trajectory and hits the middle span length of the test specimen transferring kinetic energy to it when pendulum swings to the lowest position. After hitting specimen, residual kinetic energy leads pendulum to continue moving up and the pendulum pushes a hand rolling on a dial to display the dissipated energy which can also be regard as the energy absorbed by the specimen regardless of friction and air resistance due to their small contribution to the energy balance.

In present study, the swing arm length is 750 mm leading to an impact velocity of 5.2 m/s and a stored energy of 300 J and the span of specimen support is 40 mm. The impact direction must be perpendicular to specimens’ surface. To get reliable results, at least 5 test specimens were tested for each group, and the average values of the experimental results were acquired.
RESULTS AND DISCUSSIONS

Absorbed Energy

The absorbed energies for all specimens were recorded and summarized in Table 1. It can be found that specimens in group CR absorbed the highest energy of 45.3 J among four groups of specimens due to their excellent structural integrity; and the specimens in group CS presented a lower energy absorption capacity of 32.9 J as a result of their poor integrity and stiffness in spite of their larger effective sectional area compared with group CR. Such result reveals that the energy absorption capacity of a CFRP tendon is quite sensitive to the integrity. The average absorbed energy of group CS-C and group CS-O were both 2.5 J, thus indicating that energy absorption capacity of CFRP wires were not significantly influenced by the spiral shape. Comparing the absorbed energy of group CS, group CS-C and group CS-O, it can be found that the absorbed energy of an entire CFRP strand was much larger than the sum of that of all seven wires, which is due to the composite effect of wires along the impact direction.

Table 1 The specifications and results of test specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Average length (mm)</th>
<th>Absorbed energy (J)</th>
<th>Damage pattern</th>
</tr>
</thead>
<tbody>
<tr>
<td>CR</td>
<td>100.02</td>
<td>45.3</td>
<td>delamination</td>
</tr>
<tr>
<td>CS</td>
<td>100.00</td>
<td>32.9</td>
<td>wires decentralization</td>
</tr>
<tr>
<td>CS-C</td>
<td>100.04</td>
<td>2.5</td>
<td>fracture</td>
</tr>
<tr>
<td>CS-O</td>
<td>100.02</td>
<td>2.5</td>
<td>fracture</td>
</tr>
</tbody>
</table>

Damage Patterns

Typical damage patterns, including debonding between fibres and matrix of CFRP rod, wire decentralization and fracture of CFRP strand and complete fiber fracture of individual CFRP wire, can be observed, as shown in Fig 4. It can be seen that debonding/delamination between fibres and matrix, together with partial fibres fracture, was the main damage mode of specimens in group CR. During the failure process, damage firstly occurred in the middle of specimen where was directly impacted by the pendulum, and then extended from the middle to the end region. When specimens suffered the impact load, their internal energy accumulated and afterwards matrix micro-crack initiated and expanded, which performed as debonding and delamination in the macroscopic, as illustrated in Fig 4(a). In addition, serious fibre crush appeared on the impact area due to the compression stress generated by the strong impact force on small contact area; on the rear face, the fibers fractured as a result of the large transverse deformation under impact; partial inner fibers survived owning to energy dissipation by the form of delamination and fiber fracture.

The failure mode of specimen in group CS is shown in Fig 4(b). It can be observed that the wires in specimens decentralized which was possibly due to the adhesion force weakened with the destruction of epoxy resin under impact. Moreover, the damage pattern of each wire in specimens was determined by its location in the cross section of the CFRP strand. The 3# wire was the first to be applied impact load. For the wire 1# to 3#, the support to 2#, the direct impact on 3# and the location on the centre line of impact direction, made them cannot move on that direction when they were subjected to stronger impact, so that complete fibre fracture occurred. Whilst, the epoxy resin among wires were destroyed since the deformation of wires 1# to 3#, and consequently the adhesion force and the constraints weakened, which may induce wires’(4# to 7#) movement when bear follow-up load. Therefore, partial fibre fracture was observed in 4# and 5# wire, and 6# and 7# wire survived because of the avoidance of direct contact with pendulum. Furthermore, the fracture surfaces of the wires were observed to be not parallel to
the impact direction, which was resulted from that the wires suffered from multi-directional forces transferred from surrounding wires.

Damage patterns of groups CS-C and CS-O are shown in Fig 4(c) and (d), respectively. It can be found that group CS-C and group CS-O had a similar impact damage pattern, namely complete fibre fracture. Specimens were broken into two parts and just connected by winding yarn on surface. In addition, under no multi-directional force transferred from ambient wires, the fracture surfaces of CS-C and CS-O were all parallel to the impact direction, which was different with that of wires in group CS.

Figure 4 Damage patterns of specimens

CONCLUSIONS

In the present study, the transverse impact behavior of CFRP rod and CFRP strand were investigated by Charpy impact tests. The main conclusions are given below.

(1) The energy absorption capacity for CFRP rod was 45.3 J and its main damage pattern was debonding/delamination and partial fracture of the fibers.

(2) The average absorbed energy for CFRP strand is 32.9 J; the damage pattern for an entire CFRP strand was wire decentralization and partial fracture. For each wire in a CFRP strand, the damage pattern was determined by its location in the cross section of the CFRP strand.

(3) A central wire and an outer wire of a CFRP strand have the similar damage pattern and the same absorbed energy of 2.5 J. The results indicate that energy absorption capacity of wires is not significantly influenced by the spiral shape.

REFERENCES


EXPERIMENTAL AND NUMERICAL INVESTIGATION ON IMPACT LOADING BEHAVIORS OF CFRP RETROFITTED RC BEAMS WITHOUT STIRRUPS

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ABSTRACT

Six reinforced concrete (RC) beams without stirrups were tested under static and impact loading. Two out of the six beams were not strengthened as control specimens while the rest were wrapped with carbon fiber reinforced polymer (CFRP) strips for shear enhancement. This paper discusses the impact behaviour of the CFRP strengthened RC beams, including failure modes, CFRP contribution to shear strength, and dynamic responses, such as impact force, inertia force, reaction force, and midspan deflection. A nonlinear explicit finite element (FE) analysis was conducted using software program LS-DYNA. The numerical results agree well with the experimental results in terms of midspan deflection, impact force, and reaction force. The study demonstrates that the CFRP strengthening can apparently improve the impact resistance of concrete beams without stirrup. As the CFRP reinforcement ratio increased, the failure mode under impact loading was changed from brittle shear failure mode to flexural-shear mode, even to flexural ductile mode.

KEYWORDS

CFRP, RC beams, impact loading, dynamic response, finite element analysis.

INTRODUCTION

FRP composites have been extensively applied in past three decades for externally strengthening old and deficient RC structures due to the significant performance improvement contributed from FRP. A considerable amount of research has demonstrated the excellent load capacity and ductility of FRP strengthened RC structural members under various static loads. Recently, the existing RC structure members may be vulnerable to extreme loads, such as blast and impact. The prominent properties of FRP would make the application for improving the dynamic behavior against extreme loads promising. Some studies (Erki and Meier 1999; Hamed and Rabinovitch 2005; Tang and Saadatmanesh 2003, 2005; White et al. 2001) investigated the impact behavior of FRP strengthened RC beams and mostly focused on the flexural-strength improvement. Saatci and Vecchio (2009) demonstrated that the shear mechanism affected the overall behavior of RC beams under impact loading and should be considered to accurately evaluate the dynamic performance. However, few efforts were spent on such area. Shafei and Kabir (2015); Pham and Hao (2016) experimentally assessed the shear-strength improvement from FRP under impact loading. The useful data was still far deficient in making reasonable impact-resistant design guideline.

In this study, an experimental program was conducted to study the impact behavior of CFRP strengthened RC beams without stirrups and a three-dimensional finite element analysis was performed to establish reasonable numerical method for providing much more useful data. Six specimens were designed and tested under static and impact loading. The impact test was conducted using a well-instrumented drop-weight impact machine under simply supported condition. Two of the six specimens were not strengthened as control specimens for both loading conditions, while the rest were wrapped with CFRP strips for shear enhancement.

EXPERIMENTAL PROGRAM

All the testing beam specimens had a 200 × 400 mm rectangular cross section with 362 mm effective depth and a 2400 mm clear span associated with a shear span ratio of 3.32. The longitudinal reinforcements consisted of four 16 mm diameter deformed steel bars at the bottom and two 14 mm steel bars at the top. The average concrete
Cube compressive and cylinder compressive strengths were 46.5 MPa and 36.5 MPa, respectively. The average yield and ultimate tensile strengths of steel bars were 490 MPa and 630 MPa, respectively.

The test setup under the three-point static bend loading is shown in Figure 1. During the static test, a force-control loading mode was used at a rate of 5 kN per 3 minutes. Two specimens, BS1 without CFRP strengthening and BS2 with a CFRP reinforcement ratio of 0.04%, were designed to fail in a shear mode under three-point static bend loading, as listed in Table 1. The theoretical load capacity $P_u$ associated with flexural failure and $P_n$ associated with shear failure are calculated based on the ACI 318-14 (2014) and ACI 440.2R2 (2008) as tabulated in Table 1. The value of $P_n / P_u$ less than one indicates shear failure.

![Figure 1 Static loading test setup: (a) schematic diagram and (b) overview](image)

**Table 1** Details of CFRP strips and theoretical load capacities under static loading

<table>
<thead>
<tr>
<th>No</th>
<th>Parameters of CFRP strips</th>
<th>Theoretical values</th>
<th>Exper. values</th>
<th>Failure modes</th>
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<th>$H$</th>
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<tr>
<td></td>
<td>$t_f$ (mm)</td>
<td>$w_f$ (mm)</td>
<td>$s_f$ (mm)</td>
<td>$\rho_f$ (%)</td>
<td>$P_u$ (kN)</td>
<td>$P_n$ (kN)</td>
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<tr>
<td>BS1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>272.20</td>
<td>145.02</td>
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<tr>
<td>BS2</td>
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<td>100</td>
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<td>272.20</td>
<td>201.90</td>
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<tr>
<td>BD1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>272.20</td>
<td>145.02</td>
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<tr>
<td>BD2</td>
<td>0.167</td>
<td>50</td>
<td>200</td>
<td>0.04</td>
<td>272.20</td>
<td>201.90</td>
</tr>
<tr>
<td>BD3</td>
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<td>200</td>
<td>0.04</td>
<td>272.20</td>
<td>214.66</td>
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<tr>
<td>BD4</td>
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<td>50</td>
<td>100</td>
<td>0.08</td>
<td>272.20</td>
<td>267.88</td>
</tr>
</tbody>
</table>

![Figure 2 Impact test details: (a) test setup and FRP strengthening scheme; (b) overview; and (c) instruments](image)

The schematic diagram of drop-weight impact test setup and CFRP complete wrapping scheme is presented in Figure 2 which also displays some types of instruments for acquiring dynamic response, such as impact force, reaction force, deflection, acceleration, and CFRP strain. Impact parameters including drop weight $M$ and drop...
height $H$, and some parameters of CFRP strips, such as width ($w_f$), thickness ($t_f$), center to center spacing ($s_f$), and reinforcement ratio ($\rho_f$), are listed in Table 1. One non-strengthened specimen BD1 and three CFRP-strengthened specimens BD2~BD4 were subjected to identical impact loading condition, except for BD3 with a higher impact height of 3.0 m. The ultimate tensile strength and elastic modulus of unidirectional CFRP sheets were 3561 MPa and 240 GPa, respectively. The four specimens were designed to fail in a shear mode under three-point static bend loading.

**EXPERIMENTAL RESULTS AND DISCUSSIONS**

**Static Test**

To investigate the contribution of CFRP under impact loading, the static test as control condition was necessarily conducted. As shown in Figure 3, the two specimens actually suffered from similar shear failure with a critical diagonal crack. At the initial stage of loading, several fine flexural cracks appeared in the midspan region. As the load continued to increase, the amount of flexural cracks increased and some propagated upward. When the load reached about 80% of the peak, some flexural cracks were inclined to the loading position and meanwhile some diagonal cracks developed adjacent to supports. Up to the peak load, a critical diagonal crack suddenly appeared without any warning, signifying the shear failure. For specimen BS2, the CFRP strips intersecting with the critical shear crack mostly fractured with the side segment torn away.

As illustrated in Figure 4, the load versus midspan deflection curves for specimens BS1 and BS2 show similar initial slopes and the decrement of slopes induced by cracking. Compared to non-strengthened BS1, the curve of BS2 displayed somewhat larger post-cracking slope and could rise up to a higher peak. The experimental peak loads, $P_t$, of BS1 and BS2 are tabulated in Table 1. It is calculated that the CFRP reinforcement ratio of 0.04% could contribute to an increase of the load capacity by about 81.1 kN corresponding to an increase ratio of 45.9%. The theoretical contribution from such amount of CFRP strips based on the ACI 440.2R (2008) is about 63.4 kN equal to 0.78 times of the experimental value, indicating the conservative design in such code.

**Impact Test**

The final crack patterns developed on one side-surface are marked and displayed in Figure 5. It can be clearly seen that non-strengthened specimen BD1 was subjected to severe shear damage and split into four parts under the 383 kg-weight impact with a falling height of 1.5 m. However, specimen BD2 with a CFRP reinforcement ratio of 0.04% merely developed some shear-plug cracks and support diagonal cracks without appearance of shear failure. As the ratio increased to 0.08% for BD4, even few wide flexural cracks developed in the midspan region and some fine diagonal cracks distributed along both sides. Under a high falling height of 3.0 m, specimen B3 suffered from brittle but less severe shear failure with an almost 45-degree major diagonal crack and several CFRP strip fracture. It was demonstrated that the CFRP strengthening could change the failure modes under impact loading from a shear failure mode to a flexural-shear mode and even to a flexural mode, and meanwhile improve the impact resistance against the blow with higher impact velocity.

The initial 30 ms-time histories of impact force, inertia force, and reaction force were illustrated together in Figure 6(a) for four specimens under impact loading, where the positive ordinate axis represents the downward direction of impact force but the upward direction of inertia and reaction forces. The origin of time axis is considered as the moment when the drop weight contacted the midspan surface of specimens. The inertia force was calculated from the following Eq. 1:

![Figure 3 Failure modes: (a) BS1 and (b) BS2](image1)

![Figure 4 Load-midspan deflection curves of specimens BS1 and BS2](image2)
Figure 5 Failure modes: (a) BD1; (b) BD2; (c) BD3; and (d) BD4

\[ F = \int_0^L m \dddot{u}(x, t) \, dx \]

where \( L \) is the entire span length of a specimen; \( m \) is the mass of reinforced concrete per unit length, equal to 1.80 kN/m; \( \dddot{u}(x, t) \) is the distributed acceleration, which was simplified to individual \( A_i(t) \) recorded by corresponding accelerometers shown in Fig. 3(c), assuming the same responses on symmetric locations and neglecting the effect of overhang parts at each end.

At the initial stage, the time histories of impact and inertia forces of four specimens started with similar high-frequency triangular shapes with high amplitude and short duration irrespective of different CFRP reinforcement ratio. The inertia forces show similar peaks and time lags to impact forces. Small negative reaction forces were excited together with time lags to impact forces, mainly because the pretightening compressive force generated by tightening the bolts threading cross beams was released due to the uplift tendency of both beam ends after the applied impact loading. Hence, it was acknowledged that the impact force was primarily resisted by the inertia force at the initial stage. Specimen B3 was tested with a higher impact height associated with larger impact energy twice than that of B2 leading to greater peaks of impact and inertia forces, stating that the CFRP reinforcement ratio of 0.04% could apparently enhance the impact resistance, even almost twice. As the ratio increased to 0.08%, the same impact condition produced the similar response to lower ratio with large residual impact resistance. After the initial stage, the impact force decrease to nearly zero but the inertia force rapidly increased to the opposite maximum values while the reaction force ascended to the positive peak. The similar absolute values of inertia and reaction forces demonstrated that the inertia force was mainly balanced by the reaction force at this stage. Thereafter, the low-frequency vibrations with lower amplitude and longer duration were excited again for impact forces except that specimen B1 showed another triangular shape without several oscillation possibly due to the severe failure. The impact force was approximately identical to the resultant of reaction force plus inertia force at such stage. If considering the actual direction of forces, the reaction force was basically balanced by the combination of the impact and inertia forces. At the final of impact, all forces approached to diminish but the deflection still continued to increase.

The 100 ms-time histories of midspan deflection of four specimens are shown in Figure 6(b), where the origin at time axis represents the contact moment and the positive at ordinate axis represents the downward deflection. Compared with the previous time histories of forces, the midspan deflections were not sufficiently activated at the high-frequency stage and subsequently increased rapidly to the peaks at the low-frequency damping-vibration stage. It was obvious that the control specimen B1 without CFRP strengthening yielded the greatest deformation while CFRP strengthened specimens B2 and B4 produced much smaller deflections. Specimen B3 was subjected to high impact height corresponding to high impact energy, resulting in almost half deflection of B1, though larger than that of B2 and B4. Hence, the CFRP strengthening could significantly limit the deformation of RC beams without stirrups.

**FINITE ELEMENT ANALYSIS**

The numerical analysis was performed using commercial nonlinear explicit software program LS-DYNA (2015) to establish an analytical method that can reasonably simulate the behavior of RC beams fully wrapped with CFRP strips under static and impact loading and to provide more useful data for impact-resist design.
Figure 6 Time histories: (a) impact force, inertia force, and reaction force; and (b) midspan deflection

Finite Element Model

Due to bidirectional symmetries, only a quarter three-dimensional finite element (FE) model was established and an example for impact test was shown in Figure 7. The drop weight was simplified to a cylinder with a specific mass density equal to the testing drop weight divided by its volume. The actual support systems were simulated by two support bars with only compressive stiffness. One end of two bars were connected to steel plates while the other end shown in Figure 8 was limited with all translational motions and permitted with all rotations. The FE model for static test was similar only excluding the drop weight and upper support and the load was applied at the middle nodes on top surface. A 3D eight-node solid hexahedron element, Solid 164 used to model the concrete, drop-weight cylinder, and the steel plates. Steel reinforcing bars and support bars were modeled using a two-node beam element, Beam 161. CFRP strips were modeled by a 2D four-node shell element, Shell 163. Perfect bond were assumed between concrete and steel bars or CFRP strips by sharing the common nodes. The continuous surface cap material model, MAT 159, was used to characterize the rate-sensitive nonlinear behavior of concrete. The steel rebars were modeled using a strain rate-sensitive uniaxial elasto-plastic material model, MAT 024. The plastic hardening modulus was assumed as 1% of elastic modulus. An orthotropic composite damage material model, MAT 022, was used for unidirectional CFRP strips with tensile rupture failure criteria. The discrete beam elements for support bars utilized the elastic spring-mass material model, MAT 196 input only compressive force-displacement curves. The drop cylinder and steel plates implemented the elastic material model, MAT 01, using the Young’s modulus and Poisson’s ratio of real steel material.

Figure 7 A quarter FE model of RC beam

Numerical Results

The numerical load-midspan deflection curves of specimens BS1 and BS2 are plotted with dash lines in Figure 4. It was seen that the FE model could accurately capture the initial stiffness and post-cracking stiffness, though the peak loads and corresponding maximum deflections showed some deviations.

The FE model was verified under impact loading in terms of time histories of impact force, reaction force, and midspan deflection. The numerical results are plotted with dash lines in Figures 6(b) and 8. It was seen that the FE model could well predict the tendency and duration of all curves and accurately capture the maximum
deflections with differences less than 10%, but large differences for impact force about 40% or more, and for reaction force about 30% or less. Despite the drawbacks, the FE model was mostly reasonable to be used for predicting the impact behavior of CFRP strengthened beams and for offering a feasible and effective approach to obtain considerably useful data for the design of FRP against impact load.

CONCLUSIONS

Based on the experimental and numerical results of reinforced concrete beams with or without retrofit, it is found that the CFRP strengthening can apparently improve the impact resistance if concrete beams without stirrups, and change the failure modes under impact loading from brittle shear failure mode to flexural-shear mode, even to flexural ductile mode. During the impact process, all the forces can satisfy the dynamic force equilibrium. The finite element method was reasonably well verified by the experimental results and shown to be able to obtain substantially useful data for the impact-resist design of CFRP strengthened RC beams.

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EXPERIMENTAL STUDY ON SHEAR MECHANISM OF CFRP STRENGTHENED RC BEAMS WITHOUT STIRRUPS UNDER IMPACT LOADING

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ABSTRACT

To investigate the impact behavior of reinforced concrete beams strengthened with externally bonded carbon fibre reinforced polymer (CFRP) strips, five beams including two control specimens were tested against static and impact loads. The failure modes and time-histories of impact force, reaction force, inertia force and mid-span displacement were analyzed and discussed, the shear mechanism of CFRP strengthened RC beams without stirrups under impact loading was obtained. The experimental results show that the CFRP strengthening significantly improved the impact resistance and stiffness of FRP strengthened RC beams, and can change the dynamic failure mode from shear failure to flexural failure. A discussion on inertia force verified the previous research that all the forces can satisfy the dynamic force equilibrium during the impact process.

KEYWORDS

CFRP, RC beams without stirrups, impact behavior, shear mechanism

INTRODUCTION

The adoption of fiber reinforced polymer (FRP) for the strengthening of RC structure has become a popular technique world-wide. The enhancement of load capacity and ductility under static loads has been demonstrated by extensive researchers. The terrorist attacks and accidents indicate that RC structures are possibly subjected to dynamic loading, such as blast, impact, during their service life. Some research have been conducted to experimentally study the ductility and blast resistance of FRP-reinforced concrete structures under blast loading (Banthia et al 1989; Cotsovos et al 2008; Xu and Zeng 2015). However, most of the existing research focus on the bending behavior and are only qualitative in character, and the failure mechanism of FRP-strengthened structures under impact loading has not been well understood (Erik and Meier 1999; White et al 2001; Tang and Saadatmanesh 2003; Wu et al 2009). Kishi (2002) carried out falling-weight impact tests on 27 shear-failure-type RC beams without shear rebar, and proposed a simple equation to evaluate the required static shear capacity for RC beams against impact load. Saatci and Vecchio (2009) demonstrated that the shear characteristics, mass and geometric properties of structure are important factors in resisting the impact loads. Huo (2016) observed that the impact loading has remarkable influence on the strain distribution in CFRP sheets, which may further affect the performance of FRP-strengthened structure under impact loading. Zhao (2015) observed two different failure modes of RC beams without stirrups strengthened with FRP U-wraps. The beams failed in shear failure mode when subjected to higher impact velocity and in flexural failure mode when subjected to lower impact velocity. Pham and Hao (2016) discussed the FRP contribution to shear strength under impact and verified against the ACI440.2R-08. There is still limited data to establish a reasonable evaluation method for impact resistance.

The impact behavior of CFRP strengthened RC beams without stirrups was investigated in this paper. Five RC beams without stirrups were designed and tested to failure, including two static tests and three impact tests. Three of the five beams were strengthened with externally bonded FRP sheet, which were of the same width and spacing, and the other two beams were control RC beams. The beams were subjected to only one impact except for U60-D-1, which was tested under three impacts. The different impact times was intended to observe the effect of previous damage on the shear behavior of the beams.
EXPERIMENTAL PROGRAM

All beams had the identical geometric dimensions and reinforcement layout. The beams were 150 mm in width, 300 mm in height, and 2700 mm in length. Three 16mm diameter longitudinal reinforcement bars were placed in the soffit and spanning the entire length of the beam. A 25 mm clear cover was provided between the top and the bottom beam surfaces and the bars. No transverse reinforcement for shear was provided.

All CFRP-strengthened beams were strengthened by the identical three-side FRP U-wraps, which were 60mm in width and at the spacing of 60 mm, as shown in Figure 1. For both static and impact tests, CFRP strain, concrete strain, steel strain were measured by strain gauges mounted onto specimens and the deflections of the beams were measured at the mid-span by a Dial gauge or LVDT. Five accelerometers were mounted onto the specimens to measure the accelerations during the impact-induced vibrations. The specimens were tested under simply supported conditions with a shear span of 1860 mm, leaving a 420 mm overhang at each end as shown in Figure 1. Table 1 shows the summary of the specimens and the main results of the static and impact tests.

A single layer of CFRP sheet with the thickness of 0.167 mm was used and the ultimate strength, Young’s modulus and ultimate strain of concrete at 28 days were 38 MPa. The average yield strength and ultimate strength of the reinforcement bars were 515 MPa and 636 MPa, respectively.

Table 1 Summery of RC beams without stirrups

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$w_f/s_f$ (mm/mm)</th>
<th>Weight M.(kg)</th>
<th>Impact height, $h$ (m)</th>
<th>Impact velocity, $v$ (m/s)</th>
<th>$E$ (J)</th>
<th>Max impact force (kN)</th>
<th>Max reaction force (kN)</th>
<th>$\delta_{max}$ (mm)</th>
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<td>—</td>
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<td>1740.8</td>
<td>547.6</td>
<td>208.5</td>
<td>12.0</td>
<td></td>
</tr>
<tr>
<td>U60-D-1b</td>
<td>60/60</td>
<td>328</td>
<td>0.7</td>
<td>3.24</td>
<td>1724.8</td>
<td>497.6</td>
<td>170.1</td>
<td>14.0</td>
<td>flexural</td>
</tr>
<tr>
<td>U60-D-1c</td>
<td>60/60</td>
<td>328</td>
<td>0.5</td>
<td>2.58</td>
<td>1089.1</td>
<td>320.4</td>
<td>164.6</td>
<td>12.8</td>
<td></td>
</tr>
<tr>
<td>U60-D-2</td>
<td>60/60</td>
<td>328</td>
<td>1.7</td>
<td>5.44</td>
<td>4855.1</td>
<td>650.3</td>
<td>163.9</td>
<td>55.4</td>
<td>shear</td>
</tr>
</tbody>
</table>

For both static and impact tests, the beams were tested in a three-point loading scheme. The static tests were conducted with a hydraulic jack which was used to apply a monotonic load at the rate of 5 kN per 3 minutes, as shown in Figure 2(a). Impact tests was carried out using the drop hammer which was dropped at a certain height onto the midspan of the beams, as shown in Figure 2(b). Two load cells were fixed to the support on each end of the beam in order to measure the reaction force during the impact. A steel cross beam was fixed at the top surface of the beams above the supports to prevent uplift of the beam without creating restraint moments at the supports.
during the vibrations induced by impact loads. An oscilloscopic fast data acquisition system captured the electric signals of the impact force and strain gauges at a rate of 5 millions sample/s per channel.

**EXPERIMENTAL ERESULTS AND DISCUSSIONS**

**Failure mode**

Figure 3 shows the comparison of failure modes of static test specimens and impact test specimens. As can be seen in Figure 3, two static test beams had similar crack patterns and both failed with a main diagonal shear crack formed from the loading point to the support. At the initiation of loading, it was observed that a first vertical crack appeared at the midspan of beam, then the width of the cracks increased and some new flexural cracks and diagonal cracks appeared with the increase of the applied load. Upon reaching the peak load, one of the diagonal crack propagated quickly and run through the whole beam. The strengthened beam U60-S had less number of cracks than the control beam and failed suddenly with the debonding of CFRP sheet.

For the impact tests, it can be seen that the beams failed in shear mode except U60-D-1, which failed in flexural mode and suffered more flexural cracks and less diagonal cracks. Control beam R-D failed in shear behavior and developed severe diagonal cracks, originating at the impact point and propagating downward with an angle of approximately 45 degrees, and then forming shear-plugs. Strengthened beams U60-D-1 and U60-D-2 had identical strengthening scheme but different failure modes when they were subjected to impacts form different drop height. It can be seen that U60-D-1 had more flexural cracks and less diagonal cracks compared to the U60-D-2, and the latter failed in flexural mode while the former failed in shear mode, resulting in different vibration characteristics. Besides, the diagonal cracks of U60-D-2 initiated from the supports upward to the beam top away from the impact point. The phenomenon can be attributed to the sufficient FRP strips, which were strong enough to prevent the diagonal cracks from running through the half span to the mid-span section.
Midspan deflection

The load versus mid-span deflection curves for the static test are shown in Figure 4. It is evident that beams strengthened with FRP strips were loaded to failure with increased stiffness and strength, and larger deflection. With the cracking of concrete, it was evident that stiffness of the strengthened beams was largely increased with the increase of the deflection. As compared to the reference beam, the stiffness of Beam U60-S increased by 98% and the shear capacities increased by 112%.

Figure 5 shows the time histories of the mid-span displacement. As can be seen, Beams R-D and U60-D-2 deflected from the original positions to the maximum displacement positions and back to the residual deflection positions. However, Beam U60-D-1, which behaved in a different way, deflected downward from the original positions to the maximum positive displacement positions and deflected upward to the negative maximum displacement with smaller residual deflection. This phenomenon may be due to the effectiveness of FRP-strengthening arrangement and the different impact energy, resulting in different failure mode: shear mode and flexural mode.

Impact load & reaction force

The time histories of the impact and reaction forces during the 30 milliseconds (ms) are shown in Figure 6 and details of the measured impact forces and reaction forces are presented in Table 1. It can be observed that the peak of impact and reaction responses depended on the beam stiffness and crack patterns. It can be found that the impact and reaction forces of the FRP-strengthened beams were larger than those of the RC beam when subjected to identical impact energy (U60-D-1a v.s. R-D). Besides, the increased drop height led to the increased impact force, reaction force and a change of failure mode from flexural failure to shear failure (U60-D-1 v.s. U60-D-2). It is also noted that the first impact deteriorated the stiffness of beams (U60-D-1a), and the impact force and reaction force decreased obviously when subjected to the second or third impacts.
Impact force vs. displacement relations

Figure 7 depicts the hysteretic loops of impact force versus mid-span displacement relations of beams R-D and U60-D-2. These loops are approximately triangular. The value of the absorbed energy are estimated by integrating the loop-area and it can be found that the absorbed energy increases with the increases of impact velocity. The energy ratio $E_a/E$ for Beams R-D, U60-D-2 are 0.75, 0.6, respectively, where $E_a$ is the absorbed energy and $E$ is impact energy. The energy ratio for both beams in the paper are similar to that described in Kishi et al. 2002.

![Figure 7](image)

Equilibrium of the beams

When a beam is impacted and accelerated, the acceleration of the beam caused inertial forces, which participated in the equilibrium of specimens, as demonstrated by (Banthia et al. 1989; Saatci and Vecchio 2009; Pham and Hao 2016). The time histories of inertial forces, impact force and reaction force of U60-D-1a, as shown in Figure 8, can be used to verify the equilibrium of the beams. In order to simplify the calculation of inertial forces, the assumption of linear distribution of acceleration was introduced and the inertial forces can be expressed as follows

$$I = \int_0^L \bar{m} \ddot{u}(x,t) \, dx = \bar{m} \left[ 0.23 \left( \frac{A_1(t) + A_2(t)}{2} \right) + 0.23 \left( \frac{A_2(t) + A_3(t)}{2} \right) + 0.23 \left( \frac{A_3(t) + A_4(t)}{2} \right) + \frac{0.24}{2} A_4(t) + 0.42A(5) \right] \times 2$$

Where $A_i(t)$ is the acceleration measured by the accelerometers, $i=1-5$, $m$ is the mass per unit length, giving $m=0.1125t/m$.

It can be found that the inertial force exactly balanced the impact force before the activation of the reaction force. However, thereafter the inertial force generally was larger than the summation of the impact force and reaction force. The unbalanced forces can be attributed to the complicated vibration characteristics of the impact-damaged beams, the effect of the environmental noise and concrete cracking, and the assumption of linear distribution of acceleration.

![Figure 8](image)

CONCLUSIONS

This paper investigates the shear behavior of RC beams with and without retrofit under static loading and impact loading. The tested results showed that CFRP strengthening can change the dynamic failure mode from shear failure to flexural failure. The impact resistance and stiffness of CFRP strengthened beams was improved and the strengthened beams developed less diagonal cracks but suffered more severe vibration when subjected to relatively low impact energy. It is also verified that impact force, reaction forces and inertial forces satisfy the dynamic force equilibrium when subjected to impacts.
ACKNOWLEDGMENTS

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NUMERICAL SIMULATION OF ULTRA-HIGH PERFORMANCE CONCRETE RETROFITTED WITH FRP PLATE UNDER DEFORMABLE PROJECTILE PENETRATION

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ABSTRACT

This paper demonstrates a numerical study on ultra-high performance concrete (UHPC) slab retrofitted with fibre reinforced polymer (FRP) plate subjected to steel ogive-nosed projectile penetration with striking velocity of 710 m/s. A calibrated and improved Karagozian & Case (K&C) concrete model, namely, MAT_Concrete_Damage_Rel3 (Mat_72R3), is used for UHPC, and an orthotropic linear elastic shell element with shear deformation model, namely, *MAT_ORTOTROPIC_ELASTIC, is applied to FRP plate. The depth of penetration (DOP) and crater diameter of UHPC slab retrofitted with FRP plate, as well as plain UHPC slab, are investigated.

KEYWORDS

Numerical study, UHPC, projectile, FRP, DOP, Crater diameter

INTRODUCTION

In recent years, terrorist attacks are becoming more and more frequent throughout the world. Not only does the terrorism lead to structural damage, but it seriously threatens resident’s regular life. Therefore, investigations on the resistance of construction materials against impact loading caused by projectiles have attracted much attention by researchers and engineers. Ultra-high performance concrete (UHPC) is a promising construction material with excellent material qualities, such as ultra-high strength, good ductility, excellent durability, outstanding abrasion resistance, self-consolidating workability and low shrinkage (e.g. Rossi et al. 2007; Prem et al. 2012 ). Experimental investigations on UHPC against high-velocities projectile impact have been conducted throughout the world (e.g. Sovják et al. 2013; Máca et al. 2014; Wu et al. 2015), indicating UHPC has a better impact resistance against impact loading than conventional concrete, but very limited numerical research has been carried out to investigate impact response of UHPC. Attempts have been made to build and optimize the material model for accurately numerical simulations of fibre reinforced polymer (FRP) composite (e.g. Deka et al. 2008; Nam et al. 2009; Chandekar et al. 2010). This paper presents a numerical study on impact response of UHPC slab and UHPC slabs retrofitted with glass FRP and carbon FRP plates subjected to high velocity projectile penetration.

FINITE ELEMENT MODEL VALIDATION

Material sample tests, such as uniaxial compression test, split tensile test and flexural strength test, were carried out so as to obtain the mechanical properties of UHPC slab retrofitted with FRP plate (UHPC-FRP), and to evaluate the parameters input in the finite element model for numerical simulation. In this study, uniaxial compression and four-point bending tests were conducted to validate the accuracy of the material model for UHPC used in the numerical simulation (Máca et al. 2014). In the numerical simulations of uniaxial compression and four-point bending tests, shown in Figure 1, LS-DYNA Implicit Solver was used for the quasi-static analysis. The numerical results in terms of compressive strength, modulus of elasticity, flexural strength and mid-span deflection have been obtained within 15% deviation compared with experimental results, shown in Table 1.
NUMERICAL SIMULATION OF UHPC-FRP TARGET AGAINST PROJECTILE PENETRATION

Three-dimensional numerical models of both UHPC-FRP and steel ogive-nosed projectile were built and meshed by using finite element software ANSYS. The process of projectile penetration into UHPC-FRP target was simulated and analysed by using explicit finite element package LS-DYNA. The FRP plate was employed in four-point shell elements, and the UHPC slab and steel projectile were developed as different parts using eight-node solid elements with one-point integration and hourglass control. *CONTACT_ERODING_SURFACE_TO_SURFACE was defined for the contact between the projectile and the FRP plate, as well as between the projectile and the UHPC slab. *CONTACT_AUTOMATIC_SURFACE_TO_SURFACE was defined for the contact between the FRP plate and the UHPC slab. In the numerical simulation of UHPC-FRP target against projectile penetration, LS-DYNA Explicit Solver and Lagrangian solution technique were used for the dynamic analysis.

Material models

Figure 2 (a) shows the geometry of UHPC slab with dimensions of 400 mm × 400 mm and thickness of 50 mm. *MAT_CONCRETE_DAMAGE_REL3 was used to build UHPC slab with 156 MPa compressive strength. As the K&C concrete model is only available for conventional concrete, including NSC, HSC and HPC, a K&C concrete model with manually calibrated parameters, which has been validated in the simulated static compressive test and four-point bending test, was employed in UHPC. The key inputs for UHPC employed in the simulation of projectile penetration are listed in Table 2. \( \lambda \) and \( \eta \) govern the damage function and scale factor, respectively. The relationship between \( \lambda \) and \( \eta \) is demonstrated in Figure 3, where the red solid line represents the fit for the conventional concrete, and the blue dash line represents the fit for UHPC.

Table 2 Key inputs of Mat_72R3 for UHPC

<table>
<thead>
<tr>
<th>Density (kg/m³)</th>
<th>Tensile strength (MPa)</th>
<th>Locwidth (mm)</th>
<th>( b_1 )</th>
<th>( b_2 )</th>
<th>Omega</th>
</tr>
</thead>
<tbody>
<tr>
<td>2400</td>
<td>9.9</td>
<td>1.9</td>
<td>0.82</td>
<td>2.1</td>
<td>0.1</td>
</tr>
</tbody>
</table>
Previous research shows the mechanical properties of concrete are very sensitive to strain rate, and the mechanical properties of concrete under high loading rate conditions significantly differ from those under quasi-static conditions (e.g. Rong et al. 2010; Rong and Sun 2012). Typically, the strain rate effects on both tensile and compressive strength of concrete are defined as a dynamic increase factor (DIF), which is the ratio of dynamic to quasi-static strength versus a range of strain rates. DIF for UHPC used in the present study was taken into account by conducting Split Hopkinson Pressure Bar (SHPB) tests (Su et al. 2016), and DIF values both in compression and tension are shown in Figure 4.

An automatic generated equation of state *EOS_TABULATED_COMPACTION with calibrated bulking unloading modulus was applied to reflect the volumetric stress and strain with respect to impact response caused by high velocity projectile penetration. An erosion algorithm *MAT_ADD_EROSION was also employed in the material model to simulate the localized damage of UHPC targets. Two erosion criteria with tensile stress and shear strain corresponding to material fracture and failure are defined and applied independently in the present study. Once any one of the criteria reaches the critical value, the failed nodes and elements will be immediately deleted from calculations. In this simulation, the maximum values for the shear strain and tensile stress are set as 0.4 and as double uniaxial tensile strength of the UHPC slab, respectively, according to existing studies proposed by Teng et al. (2008), Wang et al. (2010) and Liu et al. (2016).

According to previous research by Nam et al. (2009) and Pan et al. (2011), the orthotropic linear elastic shell element with shear deformation model, namely, *MAT ORTOTROPIC ELASTIC, was used to build FRP plate with thickness of 2 mm. For the dynamic behaviour of FRP, the bond effect is an insignificant parameter in the overall structure as a results of extremely frequency incident and wave governing response of the structure (Nam et al. 2009). Therefore, bond effect between FRP plate and UHPC slab can be ignored and assumed as a perfect bonding condition (Choi and Krauthammer, 2003). The material parameters of glass FRP (GFRP) and carbon FRP (CFRP) were obtained from Pan et al. (2011) and Siromani et al. (2014), shown in Table 3.

<table>
<thead>
<tr>
<th>Material</th>
<th>Density (kg/m³)</th>
<th>Elasticity modulus (GPa)</th>
<th>Shear modulus (GPa)</th>
<th>Poisson’s ratio</th>
<th>Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GFRP</td>
<td>1600</td>
<td>101.0</td>
<td>3.0</td>
<td>0.30</td>
<td>2.0</td>
</tr>
<tr>
<td>CFRP</td>
<td>1800</td>
<td>235.0</td>
<td>5.3</td>
<td>0.32</td>
<td>2.0</td>
</tr>
</tbody>
</table>
The geometry of steel ogive-nosed projectile, where the caliber-radius-head (CRH) ratio value is 3.0, is shown in Figure 2(b). The material model *MAT_JOHNSON_COOK coupled with equation of statement *EOS_GRUNEISEN was used to build the three-dimensional steel ogive-nosed projectile. The material properties of projectile were estimated by experimental data, and inputs of the material model are shown in Table 4. During the numerical simulation, the ogive-nosed projectile was launched with striking velocity of 710 m/s, to penetrate the centroid of the UHPC-FRP target.

### Table 4 Key inputs for *MAT_JOHNSON_COOK

<table>
<thead>
<tr>
<th>Density (kg/m³)</th>
<th>Shear Modulus (GPa)</th>
<th>a</th>
<th>b</th>
<th>n</th>
<th>c</th>
<th>m</th>
<th>Failure Stress (GPa)</th>
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</thead>
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<tr>
<td>9850</td>
<td>46.0</td>
<td>9E+7</td>
<td>2.92E+8</td>
<td>0.3</td>
<td>0.025</td>
<td>1.09</td>
<td>-900G</td>
</tr>
</tbody>
</table>

### Simulation data

In the present simulation, 3 shots of projectiles on the plain UHPC slab (UHPC-P) and the UHPC slabs retrofitted with glass and carbon FRP plates (UHPC-GFRP and UHPC-CFRP) with thickness of 2 mm were carried out to investigate the depth of penetration (DOP) and impact crater subjected to projectile penetration with the striking velocity of 710 m/s. Sovják et al. (2013) investigated the impact response of UHPC with 2% fibre addition subjected to the projectile penetration with the striking velocity of 710 m/s, labelled as UHPC-2-1. Máca et al. (2014) conducted the similar test on UHPC with 2% fibre addition subjected to the projectile penetration with the striking velocities of 692 m/s and 706 m/s, labelled as UHPC-2-2 and UHPC-2-3, respectively. All the data have been recorded in Table 5.

### Table 5 Response for UHPC and UHPC retrofitted with FRP plate

<table>
<thead>
<tr>
<th>Test results</th>
<th>LS-Dyna results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material</td>
<td>UHPC-P</td>
</tr>
<tr>
<td>Velocity (m/s)</td>
<td>710</td>
</tr>
<tr>
<td>Response type</td>
<td>UP</td>
</tr>
<tr>
<td>DOP (mm)</td>
<td>20</td>
</tr>
<tr>
<td>Crater diameter (mm)</td>
<td>74</td>
</tr>
</tbody>
</table>

### Results and discussion

Figure 5 demonstrates the velocity histories of UHPC-P, UHPC-GFRP and UHPC-CFRP during projectile penetration. When penetration into the UHPC slab retrofitted with glass FRP plate, the velocity of projectile reduces more rapidly, compared with plain UHPC slab. If the UHPC slab is retrofitted with carbon FRP plate, the velocity of projectile presents a greater decreasing trend. The termination time for UHPC-P, UHPC-GFRP and UHPC-CFRP targets when stopping to penetrate and starting to bounce back is 99 µs, 90 µs and 70 µs, respectively.

![Figure 5 Velocity histories of UHPC-P, UHPC-GFRP and UHPC-CFRP during projectile penetration](image)

Figure 6 presents the impact response of UHPC-2-1 (Sovják et al. 2013), UHPC-P, UHPC-GFRP and UHPC-CFRP subjected to projectile penetration. From Figures 6(b) to 6(d), the colour varying from red to blue means the UHPC target suffers more serve localized damages. Therefore, it can be seen that the plain UHPC slab...
suffers the most severe localized damages, and the UHPC slab retrofitted with carbon FRP plate suffers the least severe localized damage.

![Figure 6 Impact response of UHPC targets against projectile penetration](image)

In Table 5, the DOP and the crater diameter of UHPC-P are 20 mm and 68 mm, respectively, which show a good agreement with the average experimental results for UHPC-2-1, UHPC-2-2 and UHPC-2-3, indicating the validation of the material model proposed for UHPC. In addition, the DOP and the crater diameter of UHPC-GFRP are 13 mm and 64 mm, respectively. The DOP and the crater diameter of UHPC-CFRP are 9 mm and 56 mm. Above results illustrate that UHPC slab retrofitted with glass FRP has a better impact resistance than plain UHPC slab, and carbon FRP plate is more effective in enhancing the impact resistance against projectile penetration compared to glass FRP plate. The impact resistance will be improved by the increasing thickness of FRP plate according to previous research by Pan et al. (2011).

CONCLUSION

The present study numerically investigate impact response of plain UHPC slab (156 MPa), as well as UHPC slabs respectively retrofitted with glass FRP and carbon FRP plates subjected to steel ogive-nosed projectile penetration (710 m/s). From this study, the following conclusions could be drawn herein:

1. *MAT_CONCRETE_DAMAGE_REL3* with manually calibrated parameters can well characterize UHPC’s quasi-static behaviour under uniaxial compression and flexural tension, as well as the dynamic behaviour subjected to high velocity projectile penetration;

2. UHPC slab retrofitted with FRP plate has a better impact resistance compared with plain UHPC plate, and carbon FRP plate is more effective in enhancing the impact resistance than glass fibre FRP plate.

REFERENCES


Special Session on FRP Composites in Spatial Structures

Organizers:
Tao JIANG
Joe GATTAS
MODULAR CORELESS FILAMENT WINDING FOR LIGHTWEIGHT SYSTEMS IN ARCHITECTURE

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ABSTRACT

The paper describes novel strategies towards coreless filament winding of geometrically complex fibre reinforced building components. The research focuses on winding processes that reduce the need for formwork, allowing for the fabrication of individual one-off components made of glass and carbon fibre rovings. The component geometry and layout of the rovings is adapted to the structural loading of the system. Through the development of a modular system, the dimension of the resulting structure is not limited by the robotic fabrication set-up, which demonstrates how the coreless winding method is applicable in large scale building implementations. The paper presents two prototypes produced by modular coreless winding: First, the ICD/ITKE Research Pavilion 2013-14 investigated and demonstrated the geometric variability of the system. Second, the Elytra Filament Pavilion at the Victoria and Albert Museum in London in 2016 extended this approach towards a fully functioning roof system of significant scale. The Elytra Filament Pavilion is additionally equipped with a fibre optic monitoring system, allowing for on-line sensing of strain and temperature and illustrating the potential of multi-functional fibre systems.

KEYWORDS

filament winding, roof structure, glass and carbon rovings, robotic fabrication.

INTRODUCTION

Filament winding represents a cost effective and often-used fabrication method for synclastic composite components such as pipes, vessels or aircraft fuselages. Typical filament winding techniques require the production of a positive mould onto which the fibres are later laid. The fabrication of this mould is an elaborate process and causes waste material. In addition, the size of the core is usually limited by fabrication constraints. Thus, filament winding is often used for repetitive synclastic components of limited size and diameter. To overcome these drawbacks, a Coreless Winding process was conceived to avoid the production of a large positive core.

CORELESS WINDING FOR MODULAR STRUCTURES – ICD/ITKE RESEARCH PAVILION 13-14

Initially, the coreless winding process was used for large scale monocoque structures (La Magna et al. 2014, Reichert et al. 2014). In a second step the process is adapted to smaller individual components (Parascho et al. 2015). The aim was the development of a winding technique for modular, double layered fibre composite structures, which reduces the required formwork to a minimum while maintaining a large degree of geometric freedom and leading to a strong and robust structural system. Through the development of computational design and simulation tools, both the robotic fabrication characteristics and the structural requirements could be simultaneously integrated in the design process. A fabrication method was developed, which uses two collaborating 6-axis industrial robots to wind fibres between two custom-made steel frame effectors held by the robots. While the effectors define the edges of each component, the final geometry emerges through the interaction of the subsequently laid fibres. The fibres are at first linearly tensioned between the two effector frames. The subsequently wound fibres both lie upon and tension each other resulting in a reciprocal deformation. This fibre–fibre interaction generates synclastic surfaces from the initially straight deposited fibre connections. The order in which the resin impregnated fibre rovings are wound onto the effectors is decisive for this process and is described through the winding syntax. The specific sequence of fibre winding allows control
over the layout of every individual fibre, leading to a material driven design process. These reciprocities between material, form, structure and fabrication are defined through the winding syntax, which therefore becomes an integral part of the computational design tool.

Figure 1 ICD/ITKE Research Pavilion 2013–14 (photo: ICD/ITKE University of Stuttgart)

The effectors are adjustable to various component geometries, thus only one reconfigurable tool setup is required. Coreless filament winding not only saves substantial resources through the lack of individual moulds, but in itself is a very material efficient fabrication process since there is no waste or cut-off of fibre mats.

Figure 2 Fabrication Set-up for ICD/ITKE Research Pavilion 2013-14

The specific robotic fabrication process includes the winding of 6 individual layers of glass and carbon fibres plus an optional layer for enclosure (Fig 4). A first glass fibre layer defines the element geometry and serves as formwork for the subsequent carbon fibre layers. These carbon fibre layers act as structural reinforcement and are individually varied through the fibres anisotropic arrangement. A simplified FE Analysis, in which the
global structure is approximated as a continuous shell, gives the orientation of the stress tensors. They are transferred into fibre orientations which are finally corrected according to the constraints of fabrication (Fig 3). The generated winding syntax allows the automatic winding of the 6 fibre layers (Fig 4) including the structurally differentiated fibre layer 3.

The woven components are joined using aluminium sleeves that are integrated into the edge of the fibrous structure. These sleeves are mounted onto the winding frame before the process begins. As the fibres are placed by the robot they are wrapped around the aluminium sleeves, becoming permanently bonded to them as the epoxy resin cures. The accurate location of the aluminium sleeves allows adjacent components to be linked by steel bolts passed through pairs of the sleeves and tightened simply by hand. Due to the depth of the components the bending in the structure is converted into a push/pull tensile and compressive load. The bolts therefore carry tension and some limited shear.

In total 36 individual elements were fabricated. Each of them has an individual fibre layout which results in a material efficient load-bearing system. The biggest element has a 2.6 m diameter with a weight of only 24.1 kg. The research pavilion covers a total area of 50 m² and a volume of 122 m³ with a weight of 593 kg.
While the aim of the ICD/ITKE Research Pavilion 2013-14 was to demonstrate the geometric and structural potential of the developed fabrication method, the aim of the Elytra Filament Pavilion is to showcase how this approach could be used in a typical architectural application. The pavilion was installed in the John Madejski garden of the Victoria and Albert Museum, London, during summer 2016. It is based on the same modular concepts as described above but the fabrication process was further simplified for the use of a single robot.

The roof was configured as a planar, slightly inclined canopy supported by seven columns. The canopy was a modular structure made up of initially 37 and finally 40 hexagonal roof components. 3 components were produced on site at specific public events. The components were covered by a polycarbonate sheet to provide rain coverage and drainage. The outer dimensions of each component are identical: the depth is 40 cm and the diameter 2.40 m. After winding each component is tempered in an oven for about six hours at 80 C°. In this project the diameter of the central aperture as well as the reinforcement with carbon fibres along the edges and on the hyperbolic surface was differentiated according to the specific loading of the respective
component. For analysis a simplified model was used as in ICD/ITKE Research Pavilion 2013-14: the woven mesh is represented as a surface and the woven edge connectors are idealised to lines (Fig 6). From material testing, the characteristic capacities of the wound fibre composites were determined (Tensile Capacity $f_{tk} = 4000\text{MPa}$, Bending Capacity $f_{bk} = 1330\text{MPa}$). From these tests a design strength of $f_{td} = 445\text{MPa}$ was chosen.

Wind loading has been considered in accordance with EN1991-1-4 General Actions – Wind Actions and the UK National Annex. The site in the central courtyard is generally sheltered as a central well within a larger bluff building structure. The wind loads applied were: Maximum uplift 0.72 kN/m$^2$ along the edge, uplift on the remainder 0.46 kN/m$^2$, max downforce 0.142 kN/m$^2$.

Figure 7 Finite Element Model of the Canopy with all Columns

Two differently sized fibre composite columns were used to support the canopy. The three larger column heads around the fabrication core took majority of the horizontal wind loads (see Fig 6), while the smaller ones were connected to 10cm diameter steel tube and carried vertical forces mainly. The maximum tensile loading applied to any column is found to be 7kN. This requires an anchor to prevent uplift. A Spirafix helical anchor, common for temporary and semi-permanent structures in the UK, is used.

The maximum expected deflection of the structure under SLS loading combinations is found to be in the uplift case with maximum of 16mm uplift and 30mm downwards movement. Given the scale of the structure and the fact that these deformations will occur only in the worst case 1 in 50 year storm, this is considered to be acceptable. The structure is generally under a very low state of stress even in the most onerous loading condition due to the comparably small loads and large surface of structure across which these are distributed. At the junctions between elements and the support points for the polycarbonate covering some stress concentrations are developed. The peak value is seen as 152MPa therefore this is still acceptable compared to the 445MPa limit.

In addition to structural analysis a variety of components and columns were tested (Fig 8). The horizontal loading was increased until a bending moment as in FE Analysis of the global structure (Fig 7) was reached. In testing, as well as in analysis, buckling of the compression loaded edges proved to be the decisive failure mode. Deformation and stiffness properties were also measured. The results were used to adjust the stiffness properties of the global FE model shown in Fig 7.

Figure 8 Testing of components

The structural monitoring was achieved through optical fibre sensors that are integrated into the composite material (Gabler and Knippers 2014). This technology requires a light emitter and a reading station to be located close to the installation and permanently supplied with energy. The structural sensor fibres are equipped with
BRAGG gratings that serve as elongation sensors. If the sensors are mounted to a structurally active surface strain is measured and the stress state in the material will be computed. If the sensor is detached from the load bearing structure, e.g. placed in a tube, it measures temperature. Six components were equipped with strain and temperature sensors in Stuttgart prior to installation. The sensing chains of the components will be connected via very small so-called FP/APC plugs. Each instrumented component has four fibre chains, each with 3 strain and 1 temperature sensor. The latter is needed for the temperature monitoring and to separate the measured elongations from thermal and mechanical loading.

Figure 9 Fibre Optical Sensor Arrangement in one component (top) and Fibre Optical Sensing Chains (bottom)

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FOLDED FABRICATION OF COMPOSITE CURVED-CREASE COMPONENTS

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ABSTRACT

Curved-crease origami is a subset of origami patterns which induce a curvature into a sheet during folding. This property has led to a number of interesting applications in civil engineering and architecture, however concise geometric parametrisation and precise folded fabrication of such components remains an ongoing challenge. In this paper, the accuracy of an existing simplified geometric parameterisation for curved-crease origami is assessed by manufacture and measurement of curved-crease components. These are constructed with a new method for folding of fibre-reinforced timber (FRPT) sheet materials. A set of eleven prototypes are constructed using a simplified geometric parameterisation based on reflection of cylindrical surfaces about straight-crease origami patterns. This method was previously developed by the second author and provides a concise geometric description of curved-crease origami, but does not account for elastic or plastic bending energy. The 3D scanned prototype geometry is compared with the simplified design geometry, and it is found that the folded fabrication method is able to produce a consistent part and that variation between manufacture and design geometry is acceptable for applications where global form is critical, such as decorative components, but not for those where centreline geometry is critical, such as structural components.

KEYWORDS

FRP, curved crease origami, folded fabrication.

INTRODUCTION

Thin materials can be folded to create geometries with excellent structural performance characteristics, for example buckling resistance or bending capacity. The historical art of origami is rich with knowledge of material folding and thus origami-inspired engineering is concerned with application of novel folded geometries for new types of engineering structures and devices. Curved-crease origami is a subset of origami concerned with curved-line folding, which imparts a curvature in the sheet during folding (Demaine et al., 2011). This curvature is a striking feature which had led to successful application of curved-crease origami in many different field of design. In mechanical engineering, curved folds have been used to fold a car shell (Kilian et al., 2008), a deployable compliant rolling-contact element joint (Nelson et al., 2015), self-assembling propellers (Miyashita and Igarashi, 2015), and high energy-absorbing crash box and sandwich panels components (Garrett et al., 2016, Gattas and You, 2015).

In architecture and civil engineering, applications of curved-line folding are seen across a range of scales and materials. Modular or tessellated components have been used for kinetic façades with adaptive shading capability (Vergauwen et al., 2013) and robotically-fabricated sheet metal assemblies (Epps and Verma, 2013). Novel spatial structures have also been developed including polyhedra skeletons (Chandra et al., 2015a) and vault structures (Tachi, 2013). In terms of those with a demonstrated structural performance benefit, curved folded plate beams built with timber (Buri et al., 2011) and fibre-reinforced timber (Hansen et al., 2016) have been seen to have a substantial increase in strength and stiffness compared to convention thin-walled sections.

The above applications have all been enabled by extensive preceding work in mathematics, modelling, and simulation of curved-line folding. Pioneering work by Huffman (1976) has led to the modern design methods for curved crease origami geometry (Demaine et al., 2011; Koschitz et al., 2008). These include physical methods and computational methods. Physical methods include the mirror reflection method based on inversions of developable surfaces about truncation (Mitani and Igarashi, 2011), or manual tracing of physical prototypes (Tachi, 2011). Computational methods typically involve generation of a smooth curved developable surface from planar quadrilateral (PQ) strips, optimised for minimum discrete bending energy in curvature while enforcing developability (Kilian et al., 2008). Such methods have been extended to enforce exact developability at every
folding step (Solomon et al., 2012), to enable freeform geometries (Chandra et al., 2015b), and to convert known straight-crease origami into curved-crease variants (Gattas and You, 2014).

The mathematics and application of curved-crease origami thus has high potential for the creation of new and beneficial structural forms. However, concise geometric parametrisation and precise folded fabrication of such components remains an ongoing challenge, with diverse methods available for both but no benchmarking of their relative efficacy. This paper aims to assess the accuracy of an existing simplified curved-crease origami geometric parameterisation with manufacture and measurement of composite curved-crease components.

**GEOMETRY, MANUFACTURE, AND MEASUREMENT**

**Geometry**

A modular curved-crease origami component was designed using the simplified design method proposed in Gattas and You (2014). In this method, bending energy is neglected so that a curved-crease parameterisation can be developed as an extension of a straight-crease origami parameterisation. It can be understood as follows. First, consider a simple corrugated fold shown in Figure 1(a). It is folded from a square of paper with the crease pattern shown in Figure 1(d) and defined with three parameters; side length $s$, dihedral angle $\theta$, and corrugation proportion $l'=l_1/l$, where $l=\sqrt{2s}=2(l_1+l_2)$. This can be extended by introducing a reverse fold as shown in Figure 1(b) and (e), with an additional parameter $\phi$ sufficient to uniquely define the location of this fold. New parameters can be shown as dependent, with $l_2=l_1/\sin \phi$ and $\eta_1$ and $\eta_2$ related to $\theta$ with equations given in Gattas and You (2013). The final curved crease module, shown in Figure 1(c) and (f), is then created by specifying a gradient parameter $\varphi$ to define an elliptical curved through the three points of the straight crease reverse fold and projecting this along reflected straight-crease axes. An any specified elliptical curve will form the intersection of some cutting plane and a cylindrical surface, so the generated projected surface forms a reflected conical surface and is thus developable. The final curved-crease element is therefore concisely defined with five parameters: $s$, $\theta$, $l'$, $\phi$, and $\varphi$. For the present paper, modules are used with $s=300\text{mm}$, $\theta=109.3^\circ$, $l'=0.15$, $\phi=66.8^\circ$, and $\varphi=15.5^\circ$.

![Figure 1](image_url)

**Manufacture**

Manufacture of the curved-crease panels was achieved with a recently developed cured-in-place composite manufacturing processing (Hansen et al., 2016). The process takes a flat sheet of fibre and places a slow-curing resin at fold line locations and a fast-curing resin elsewhere. The differential curing time allows a 3D part to be precisely folded after the fast cure resin is set, but prior to slow cure resin set. This process is followed as shown in Figure 2(a) and (b), with a 0.15-0.25mm thick Biotex Flax 100g/m2 2x2 Twill style weave natural fibre bonded with Amperg22 Normal and Slow Epoxy, with 4 and 30-hour cure times, respectively, under vacuum and 20°C
room temperature environment. Silicon dioxide (SiO2) powder was added to the slow cure resin to increase the resin viscosity and thus improve fold line accuracy by minimising resin bleed. A 0.6mm thick Spotted Gum timber veneer was also bonded, shown in Figure 2(c), again for more precise control of folded hinge width than is otherwise possible with FRP alone. Orientation of the grains on the timber veneers was perpendicular to the proposed fold direction to prevent the cracking in folding state. In the foldable state, the fold lines acted as hinges and panel surfaces were elastically bent into their final shape and placed in a jig. Once fully cured, they were held this shape unassisted, Figure 2(d). Final thickness of the cured samples was measured as approximately 1mm.

Figure 2 Manufacture procedures. (a) Slow cured resin in paste applied on the fibre sheet fold locations. (b) Fast cured resin applied elsewhere. (c) Application of timber veneer. (d) Folded and fully cured prototype.

Assembly and Measurement
A plywood timber backboard was used to mount 11 component panels in a modular hexagonal grid as shown in Figure 3. Panels were fixed by fitting triangular flaps into pre-cut slots on the backboard, ensuring precise relative positioning of all panels. The completed assembly was then 3D scanned, with the surface of the panels scanned individually relative to a constant reference plane in space. A FaroArm 3D scanning system was used, which is accurate to between 0.024mm and 0.064mm.

Figure 3 Assembled components. (a) Front elevation. (b) Front perspective. (c) 3D scanning in progress.

RESULTS AND DISCUSSIONS
Global and Local Surface Error
The 3D scan process provided surface positioning data for each of the 11 panels separately, but with a preserved global reference plane. A ‘global’ measure of surface error was obtained by re-assembling 11 scanned reference planes into a single structure and compared globally with the virtual design model, Figure 4(a) and (b). The surface scan data was divided into approximately 97k data extraction points using Rhino/Grasshopper parametric CAD tools, located on anchor points on the design backboard, and the closest distance between the scanned and designed surfaces automatically extracted, see Figure 4(c). A ‘local’ measure of surface error was obtained by comparing individual module scan data to design geometry, Figure 4(d) and (e). Scan data for each module was divided into approximately 40k data extraction points and re-positioned manually to give an approximate best-fit with anchor positions of the virtual geometry. The closest distance between the scanned and designed surfaces was again automatically extracted, as shown in Figure 4(f).

Consistency of Manufactured Panels
Global and averaged local geometric analyses gave a maximum difference between manufactured and design surfaces of 3.79mm and 3.34mm, respectively. This similarity indicates that assembly procedures were accurate. Considering individual module results, shown in Figure 5, all modules showed geometric variation between 2.1% and 3.7%, with percentage values obtained as the maximum absolute difference between manufactured and design geometry, relative to a design radius of 120.3mm. This shows reasonable consistency in the manufacturing method. Physical inspection also showed two trends in where this variation occurred: as a
flattened curvature form and as an outward tilt of corner ends, see Figure 6a and b, respectively. The former is attributed to the neglecting of bending energy in the simplified geometric parametrization and is discussed further in the next section. This latter will be investigated in future papers.

Figure 4 Geometric analyses. Global analysis (a) 3D scanned geometry, (b) 3D scan aligned with design geometry, (c) maximum displacement. Local Analysis (d) 3D scanned panel, (e) 3D scan aligned with design geometry, (f) maximum displacement.

Figure 5 Difference between prototype and design surface geometries.

Figure 6 Examples of observed local geometric variations (a) flattened curvatures and (b) outward tilt at corners.

Comparison of Manufactured and Simplified Design Geometry
The manufactured panels were observed to be relatively consistent in their form and so a comparison of ‘averaged’ manufactured geometry versus design geometry was undertaken to assess the efficacy of the simplified curved-crease design method. Of primary interest is panel curvature, as the simplified method neglects bending energy
considerations in formulating the curved-crease definition. Therefore, to analyse this principal effect, a cross-section analysis was used to extract the curvatures of all 11 manufactured panels at four different locations: two on convex outer surfaces and two on concave inner surfaces as shown in Figure 7a. The extraction plane locations were selected based on those which provided the longest length along the curved geometry and were perpendicular to folded axes. Cross sections were measured in terms of their variation from the constant design radius of 120.3mm and at 19 points, 10 degrees apart, along each curve as shown in Figure 7(b) and (c).

Results are summarised in Figure 8 and it can be seen that, on average, both outer and inner curves have a smaller radius than the design radius toward the centre of the panel and a larger radius towards the outside of the panel. This behaviour is attributed to the assumption of constant curvature made in the simplified design method. Consideration of bending energy, particularly for a module with non-uniform cross section area such as the present one, is therefore critical in prediction of accurate predictions of elastically-bent curved crease components. It should be noted though that the variation shown in Figure 8 is taken relative to the design radius and is relatively small at less than ±4%. However, variation measured relative to the plate thickness would be two orders of magnitude higher. The simplified method can therefore be considered appropriate for applications where global form is critical, such as decorative components, but not for those where centreline geometry is critical, such as structural components.

CONCLUSIONS

This paper has demonstrated an effective manufacturing method for the production of curved-crease components with a consistent surface geometry. It has also found that a simplified design geometry used to model curved-crease components, which does not account for elastic or plastic bending energy, was able to give a prediction of the manufactured surface geometry to within ±4% of design radius, or ±400% of component thickness. This indicates the geometric and fabrication methods are acceptable for applications where global form is critical, such as decorative components, but not for those where centreline geometry is critical, such as structural components.

ACKNOWLEDGMENTS
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ABSTRACT

The article presents a digital design and construction system for weaving structure that integrates architectural and structural design. Influenced by architects such as Zaha Hadid and Frank Gehry, a lot of contemporary architectural designs employ a great deal of digital technologies, and explore complex spatial expression and construction of organic curvilinear forms. Despite large amounts of effort that is dedicated into study and optimization of such design, its construction is still expensive. Supported by the contemporary digital technologies, the following study proposed a new weaving structure system, which is composed of long elastic members that are weaved together. The new structure system can be used in construction of continuous curving surfaces of complex topologies. The benefits of this system are as follows: Firstly, it is a design and construction system that has a wide adaptation to different curvilinear geometries. Secondly, the final form is developed from the counterpoises between interacting forces of the weaving elastic members; therefore the result of this structural form-finding process is more rational comparing to a designed geometry that has not been structurally optimized. Thirdly, it greatly reduces the tedious three-dimensional location and fabrication work in the construction of complex curvy surface, and significantly reduces the construction cost.

KEYWORDS

Architecture-structure integrated design, weaving structure; free-form surface; remeshing; FRP.

INTRODUCTION

In recent years, we have witnessed that contemporary digital technologies are exerting a profound impact on architectural design. With digital technologies, the process of architectural creation is no longer a mere organization of space, form and function, but a more comprehensive process of interdisciplinary performance optimization. The creative design of novel building systems with the integration of architectural design, structural engineering and building construction is one of the most important research directions in the field of architecture.

The development of digital technologies equips architects with parametric and generative design capabilities. With the help of 3D modelling software and scripts, quite a lot of unprecedented organic complex forms such as the Taichung Metropolitan Opera House or the Yokohama Port Terminal, were therefore invented or discovered, thus greatly expand the capabilities of architectural design. The structural system and the construction methods used in some projects, nevertheless, do not correspond with its continuous organic shape, which results in undesired difficulty and cost, and obscure the expression of design conception. Hence, given the new trend of architectural design and construction, it is necessary to propose a reasonable structural system which can highly adapt to different organic complex forms.

This paper proposed a new weaving structure system for design and construction. The aim of this research is to develop a holistic system for design, tectonics and construction that integrates architectural design and structural engineering together. The design and construction of the interweaving structure is entirely based on the digital platform. Parametric and generative design methods, structural simulation etc., have been combined to develop the form-finding algorithm, and a construction optimization and supporting program is developed to organize the construction work. Finally some experimental physical weaving structures are constructed to test the system.
RELATED WORK

Researches in several aspects including weaving structure, mesh grid optimization and digital construction method are related to the present work. The following is a brief overview of previous related works.

Not much research has been found on weaving structure in architecture or structural engineering related areas. Some essays mentioned the keyword “weaving structure” or “woven structure” but they don’t correspond to the architectural implication. For example, Toshihiko Okumura et al. (1995) proposed a 3-D woven fabric composite, and for Krasimir Yordzhev et al. (2012) “weaving structure” is a mathematical term referred to combinatorics. Some essays from Chinese researchers focused on “woven building skin” (Yu et al., 2014) “tectonic in woven form” (Cai, 2013) or “woven surface” (Wang, 2010) while in fact most of them are discussing some forms which merely look like weaving. There are some completed projects whose organic envelopes are constructed with interconnecting continuous structural members, such as Haesley Nine Bridges Golf Club House designed by Shigeru Ban Architects, and the Phoenix International Media Center in Beijing designed by BIAD. These works are not the weaving structure we propose since they do not make use of the friction and pre-stress in the bending members. Feng (2007) proposed a FRP woven web structure in which tensile FRP strips are woven into flat roof structure, but the study does not utilize the bending resistance of the members for complex geometry.

Mesh model plays an important role in the optimization method of spatial grid during the generation process of weaving structure. There are considerable related works that inspire us. In the context of computer graphics, mesh optimization is to treat the position and connectivity of vertices with given global functions in order to greatly reduce the number of the vertices and faces. For example, Hoppe et al. (1993) defined this global function as an Energy Function. The resulted mesh grid may be irregular, and a following operation of mesh relaxation is needed (Ohtakey et al., 2001). Meanwhile, in the realm of digital fabrication and construction, some optimization algorithms are well developed. Unlike those mentioned above, the algorithms are used for panelization by optimize the panel segmentation of curvy building façades, generally aiming to equalize the edge length and planarize the mesh faces. Some of the works include the panelling method by Eigensatz et al. (2010), and Area-Equalizing Remeshing by Botsch et al. (2004). These methods are designed exclusively either for speeding up calculation or for the optimization of panels, and they are not suitable for generation and optimization of mesh for weaving structure.

A lot of researches that uses digital tools in fabrication and construction of double curved surfaces of architecture have been carried out. Inspired by the bird’s nest weaving technique, Giulio Brugnaro (2015) manages to construct a weaving grid by rattan using a numerical controlled robotic arm. The work employed an interactive and dynamic digital construction process, resulted in a deformed form with uneven lattice, which is different from the geometrically and structurally optimized grid of our research. But it could be seen from the research that the weaving structure proposed in this paper is highly likely to be constructed by numerical controlled tools.

WEAVING STRUCTURE

Weaving is generally the technique with which the line-like members can be bent and interweaved into 2D or 3D forms. Traditional weaving handicrafts adopt bamboo strips (Fig. 1, left) or rattan. They are useful, economical and artistic, plays an important role in people’s daily life. These weaving structures take advantage of material strength and stiffness: the interweaving rods are dense and bound by the friction between each other that constitute a self-balanced pre-stressed structure system. Although these handicrafts are usually of simple and symmetrical shape such as bottle or basket, weaving technique does have potential in the making of more complex form (Fig. 2, left).

Figure 1 a bamboo tea caddy (left), a design of complex weaving structure (right)
Weaving structure is a kind of spatial structure such as Buckminster Fuller’s geodesic dome. However, the weaving structure makes use of long continuous members rather than short segmented tubes that are connected by rigid or hinge joints as found in many spatial structures (Fig. 2 left). Compared to that, the weaving structure harnesses the pre-stress force inside the bent continuous members to be a self-supporting unity (Fig. 2 right). Since the members can bear bending force, it is possible for the weaving structure to form more complex geometry comparing to pure tensile system such as membrane, and pure compression system such as dome.

Figure 2 spatial structure with bolt ball joints (left) and weaving structure (right)

This paper discusses the overall design and construction process of the weaving structure. Firstly, the 3D digital model of a previously designed geometry is converted and optimized into a mesh model particularly for weaving structure with designated edge length. Then the model is further converted into the model of weaving structure with continuous rods. Afterwards, this structure model is verified and optimized through structural simulation. Later, the construction material and joints are studied, together with a digital program that generate the construction drawing, optimize the workflow, and support the construction process. Finally construction work that requires collaboration of multiple people is carried out. Collaboration is necessary because the weaving order is more complex than that of simple geometry such as bottle or basket.

SPATIAL GRID GENERATION THROUGH REMESHING

It is well known that there are mainly two kinds of mathematical models used to describe almost any 3D shapes: NURBS surface model or polygon mesh model. The advantage of NURBS is that it can describe curved surface smoothly and accurately with limited amount of data, while mesh is good at its flexibility in topology and geometrical similar to weaving lattice for our research. Mesh model is selected for the generation and optimization of the grid of weaving structure, because the aim of weaving structure is to facilitate construction of curved form with complex topology, and mesh edges can be transformed into weaving rods with vertices into joints conveniently.

The weaving structure, nonetheless, has additional requirements for the mesh model: Firstly, the size of each mesh face should be similar to regular triangle with edge length restricted within a certain range. This is an essential step to ensure the structure stability, controlled rod curvature, as well as the formal aesthetics. Secondly, for every vertex in the mesh, the number of edges it connects to should be taken into consideration. Since the actual weaving member is of a certain diameter rather than zero thickness as in the mesh model, too many members around one joint are doomed to unwanted thickness and deformation. Thirdly, it is obvious that the long and continuous weaving members are conducive to the weaving structure. As to one specific vertex, it can be seen that if it connects 2n edges (n is in positive integers), then it can be converted into a joint with n continuous members. If it connects 2n+1 edges, then apart from n continuous members, there will be one additional rod which starts from the joint. Apparently the more loose end there are, the worse the structural integrity will be.

Technically, these problems can be solved based on the Mesh Class referred to the Rhino SDK C# documentation. Before the operation, a constant is set for the standard edge length, which controls the scale of the lattice. Then, an iteration of two operations is performed to reorganize the mesh model.

Insert Vertex

If a mesh edge is much longer than the standard length, it is supposed to be divided into more edges. The operation of ‘insert vertex’ deals with this requirement that traverse the mesh structure and adds all the edges in a list sorted by their length in descending order. For the edges longer than the standard length, if the edge is longer than 1.5 times of the standard edge length, then a vertex is added on the edge and the adjacent faces are split accordingly. It is also viable under non-manifold circumstances (Fig. 3). By this way, in terms of multi-threaded environment, it needs to make sure this method is atomically adding point, i.e. these edges being operated upon should not be interfered by other threads.
**Remove Vertex**

Similarly, there could be a lot of mesh edges that are much shorter than the standard length, so some of the vertices should be removed from the mesh model. If the length is shorter than 0.7 times of the standard edge length, then the two end point of the edge would be merged into one vertex in the middle (Fig. 4). It is easy to conclude that it is also true in the non-manifold situations.

Through the application of these two operations, the original mesh model has been adjusted into a new mesh model. The new model maintains similar spatial form, but the optimized mesh faces’ edge length is closer to the standard length (Fig. 5). The new mesh grid is better aesthetically, and suitable for a weaving structure. Minor manual adjustments are performed to adjust the small flaws on the grid.

**EXTRACTION OF WEAVING MEMBERS**

Weaving structure is composed of continuous rods; therefore, it is necessary to connect the small mesh edges into long continuous polylines that represent the weaving rods. In a weaving structure, the rods should be as straight as possible in order to reduce the curvature, and therefore the corresponding bending stress. The searching and connecting process is as follows: The angles between edges around each vertex are evaluated sequentially, if it is larger than a certain threshold (e.g. 135°), then the two edges of the angle will be connected (Fig. 6, left). Different continuous members are marked with various colours in the model (Fig. 6, right).

**MATERIAL SELECTION AND CONSTRUCTION EXPERIMENT**

Material selection of the weaving structure should consider the strength and deformation capacities. On one hand, the rod should be strong enough to bear various loads, including the pre-stress in the bent rods. On the other hand, the rod should own a large permissible deformation in order to achieve the required curvature. Furthermore, the rod stiffness should not be too small to maintain the natural configuration; meanwhile, the rods should be flexural for construction. If weaving structure is used in construction of small structures, such as pavilions or exhibition facilities, the components should be moulded by hands. A variety of materials had been considered, including bamboo rods, plastic pipes, and carbon or glass fibre reinforced plastics (FRP). In recent experimental construction
tests, glass FRP rod is chosen as the construction material after comparison of several materials. FRP has enough strength for construction, favourable elasticity for bending, and better mechanical characteristics than bamboo. FRP rods have different diameters that allow us to easily choose the proper stiffness.

The interaction of weaving rods and structural form-finding are simulated by a physical simulation engine in the Rhinoceros Grasshopper platform called Kangaroo. The system for simulation is composed of springs that could bear tension and compression, bending resisting hinge that could bear moment force, and particles that could bear gravity and other forces. Although Kangaroo is a simple physical engine, which cannot provide a detailed mechanical simulation, it is enough for the qualitative analysis and optimization of the weaving structure during the design phase.

![Figure 7 Conceptual simulation using Kangaroo (left: axial force, right: bending moment)](image)

Different types of joints are tested including hot melt adhesive (HMA) and nylon cable tie. Nylon cable tie which allows the joints to rotate is chosen, as it creates enough frictions between the weaving rods. Besides, nylon cable tie is also convenient for fabrication in construction.

Several test models have been constructed via FRP rods, whose diameters range from 1.5 mm to 3 mm as shown in Fig. 8. The whole system is proven to be feasible via the design and construction process of the testing models. The experiment also exposed some problems that requires further research. One challenge is that, since the rods with large curvatures are sharply bent, they are able to fracture, such as the three tip circles in the left model shown in Fig. 8. This phenomenon suggests other advanced materials should substitute for these rods, or special methods should be employed to reinforce these rods.

![Figure 8 Experimental models for the weaving structure](image)

**CONCLUSIONS**

Inspired by a variety of traditional handicrafts and supported by contemporary digital technologies, a lightweight weaving structure system is developed aiming to facilitate the construction of complex organic architectural form. This structure system is composed of long continuous rods which are interwoven into a structure entity. The pre-stress force in the bent rods increases the integral stiffness of the structure and gives full play to its material advantages, including light weight and high tenacity. The weaving structure can be used in construction of complex curvilinear forms at a relative low cost. From the construction experiments, this weaving structure system has been proven to be effective and can be applied to light-weight indoor and outdoor structures for exhibition, leisure and recreation.
Due to the complex topology, each woven rod has a complex flowing direction. For the present test, it has to be constructed through collaboration of a big research group. Considering the rapid development in digital fabrication and construction, it might be possible that the weaving structure could be constructed on-site by the numerical controlled robotic arm in the near future.

In conclusion, it is important to note that although the weaving structure is supposed to be used for light-weight structures, the research findings such as form-finding rule of the spatial grid, the design of the joints and digital construction methods, could also provide ideas for large scale structure systems.

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FORM-FINDING OF BENDING-ACTIVE GRIDSHELLS IN FRP COMPOSITES

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ABSTRACT

Bending-active gridshells are formed through active bending of a mat of planar grids assembled from long continuous straight members connected by scissor-like joints that are free to accommodate in-plane rotation. Their unique formation method and simple nodal connection form enable FRP composites to be their appropriate construction material. This paper proposes a form-finding approach for bending-active gridshells formed from FRP tubes within an explicit dynamic framework. The proposed approach treats the target surface geometry of the gridshell as an input into the form-finding procedure. It allows each node to have six degrees of freedom and couples all nodal degrees of freedom except the in-plane rotation for the accurate simulation of the behaviour of the scissor-like joints. Numerical verification is presented to demonstrate the effectiveness of the proposed form-finding approach.

KEYWORDS
FRP, bending-active gridshells, form-finding, explicit dynamic method.

INTRODUCTION

Bending-active gridshells (referred to as gridshells hereafter for brevity) are a particular type of grid structures that are formed through active bending of a mat of planar grids assembled from long continuous straight members connected by scissor-like joints that are free to accommodate in-plane rotation (Figure 1). The concept of gridshells was initially proposed by the famous German engineer Frei Otto half a century ago. Despite their obvious advantages including uniform members/joints, simple connection and the capability of complying with specific free-from surface geometries, only a very small number of gridshells exist worldwide, among them the Multihalle of Mannheim in Germany (Happold and Liddell 1975) and the Downland Museum in the UK (Harris et al. 2003). The majority of existing gridshells were formed from timber laths. The use of pultruded FRP tubes instead of timber laths is a recent and an important advance in the state-of-practice of gridshells (Douthe et al. 2006, 2010; Baverel et al. 2012) because: 1) pultruded FRP tubes have a much higher elastic modulus/strength than timber laths and can readily be manufactured to the desired length; and 2) the unique nodal form of gridshells overcomes the difficulty encountered in the connection of FRP tubes.

An important factor that limits the engineering practice of gridshells is the complexity of their form-finding, which in more specific terms, is the issue of finding the corresponding planar grids configuration that can accurately be deformed into the target surface geometry proposed by architects. The majority of existing form-finding methods for gridshells (e.g. Happold and Liddell 1975; Harris et al. 2003; Douthe et al. 2006; Kuijvenhoven and Hoogenboom 2012; D’Amico et al. 2014) are only capable of finding the deformed grids geometry as an output corresponding to pre-defined planar grids configuration. Even those few methods which treat the target surface geometry proposed by architects as an input and in turn output the corresponding planar grids configuration are mainly subjected to the following two deficiencies: 1) incomplete account for nodal degrees of freedom (DoFs); and 2) coupling of nodal DoFs for the accurate simulation of the behaviour of the scissor-like joints is not properly treated or clearly reported. To overcome these deficiencies, this paper proposes an improved and more general form-finding approach for gridshells within an explicit dynamic framework. The
proposed approach treats the target surface as an input and is applicable to free-form surface geometries. It allows each node to have six DoFs and couples all nodal DoFs except the in-plane rotation to simulate the behaviour of the scissor-like joints.

**GENERAL PROCEDURE OF PROPOSED FORM-FINDING APPROACH**

The proposed form-finding approach comprises five steps (Figure 2) as described below.

**Step 1**: Define the target surface geometry and place the mat of planar grids underneath. The target surface is treated as a rigid geometrical constraint. The mat must have an area larger than the area of the target surface and may have an arbitrary initial shape.

**Step 2**: Apply a fictitious pressure field to the planar grids to force them deform into the target surface geometry until all nodes of the grids inside the target surface are at rest on (in contact with) the target surface.

**Step 3**: Truncate the part of mat of grids that is outside the target surface and fix the so generated end nodes to form the boundary of the gridshell.

**Step 4**: Remove the fictitious pressure field and the rigid surface constraint (contact force) to achieve the final form of the gridshell.

**Step 5**: Generate the configuration of the planar grids in pursuit by releasing the pre-stresses in the final form of the gridshell derived in Step 4.

**EXPLICIT DYNAMIC METHOD FOR 3D BEAM ELEMENTS WITH SIX DOFS PER NODE**

**Motion Equations**

The explicit dynamic method employed in the proposed form-finding approach discretizes a gridshell into a set of particles (nodes) linked by three-dimensional beam elements. The terms *particles* and *nodes* are used
interchangeably in this paper as the particles are also the nodes of the beam elements. Each node of the beam element has six DoFs, including three translational DoFs and three rotational DoFs. The deformation of the gridshell can then be described by the motions of the particles. The motion of an arbitrary particle $\alpha$ follows the Second Newtonian Law,

$$m_\alpha \ddot{x}_\alpha = \Gamma^\text{ext}_\alpha - \Gamma^\text{int}_\alpha - \Gamma^\text{dmp}_\alpha$$

(1a)

$$I_\alpha \ddot{\phi}_\alpha = M^\text{int}_\alpha - M^\text{ext}_\alpha - M^\text{dmp}_\alpha$$

(1b)

where $m_\alpha$ ($I_\alpha$) is the mass (moment of inertia matrix), $\ddot{x}_\alpha$ ($\ddot{\phi}_\alpha$) is the translational (angular) acceleration vector, $\dot{x}_\alpha$ ($\dot{\phi}_\alpha$) is the translational (angular) velocity vector, $x_\alpha$ ($\phi_\alpha$) is the position (rotation) vector and $\Gamma^\text{ext}_\alpha$ ($M^\text{ext}_\alpha$) is the external force (moment). The damping forces $\Gamma^\text{dmp}_\alpha$ ($M^\text{dmp}_\alpha$) are introduced to damp the gridshell to a static equilibrium state, where $\mu$ is the damping factor which can be assigned with a fictitious value as the form-finding issue under consideration is static in nature. $\Gamma^\text{int}_\alpha$ ($M^\text{int}_\alpha$) is the summation of the internal nodal force (moment) exerted by the elements connected with particle $\alpha$.

Based on the central difference method, the velocity and the acceleration of the particle can be expressed by time and displacement

$$x^{n+1}_\alpha = (x^n_\alpha - x^{n-1}_\alpha) / (2\Delta t)$$

(2a)

$$\dot{x}^{n+1}_\alpha = (x^{n+1}_\alpha - 2x^n_\alpha + x^{n-1}_\alpha) / \Delta t^2$$

(2b)

$$\phi^{n+1}_\alpha = (\phi^{n+1}_\alpha - \phi^{n-1}_\alpha) / (2\Delta t)$$

(2c)

$$\ddot{\phi}^{n+1}_\alpha = (\dot{\phi}^{n+1}_\alpha - 2\dot{\phi}^n_\alpha + \dot{\phi}^{n-1}_\alpha) / \Delta t^2$$

(2d)

Substituting Eq. 2 into Eq. 1 yields

$$x^{n+1}_\alpha = c_1 \Delta t^2 (\Gamma^\text{ext}_\alpha - \Gamma^\text{int}_\alpha) + 2c_1 \dot{x}^n_\alpha - c_1 c_2 x^{n-1}_\alpha$$

(3a)

$$\phi^{n+1}_\alpha = c_1 \Delta t^2 (I^\text{int}_\alpha - I^\text{ext}_\alpha) + 2c_1 \dot{\phi}^n_\alpha - c_1 c_2 \phi^{n-1}_\alpha$$

(3b)

where $c_1 = 1 / (0.5 \mu \Delta t + 1)$, $c_2 = 0.5 \mu \Delta t - 1$, $\Delta t$ is the time increment. Eq. 3 allows the displacement of particle $\alpha$ at the current time step to be explicitly derived from known information of previous time steps. More details of the framework of the explicit dynamic method and the formulation of the internal force of the three-dimensional beam elements can be found elsewhere (Yu and Luo 2011a, 2011b).

Nodal Coupling

In a gridshell, the continuous profiles are connected by scissor-like joints which allow free in-plane rotation during the formation process. The behaviour of planar scissor-like joints has been successfully modelled in a previous study (Yu and Luo 2009). However, their behaviour is more complicated in three-dimensional space. As shown in Figure 3, profile $AJB$ and profile $CKD$ connect with each other by node $J$ and node $K$. In the numerical model, these two nodes share the same coordinates. Therefore, the motions of node $J$ and node $K$ possess the following features: 1) their translations are coupled at any time; 2) their out-of-plane rotations are coupled at any time; and 3) their in-plane rotations are independent at any time.

![Figure 3 A scissor-like joint in three-dimensional space](image)

The two profiles are numerically represented by four beam elements (beams 1 to 4 in Figure 3). Since the translations of node $J$ and node $K$ are coupled at any time, these two nodes can be merged into one in the calculations of translations and their nodal masses can thus be expressed as

$$m_{\text{merge}} = m_J + m_K$$

(4)

So are the nodal internal forces and external forces...
For the convenience of dealing with rotational coupling, a tangent plane $\pi$ is defined with a tangent coordinate system $\tilde{\Omega} = \{ \tilde{e}_x, \tilde{e}_y, \tilde{e}_z \}$, where $\tilde{e}_i$ is averaged from the cross products of the four pairs of neighboring beam elements and $\tilde{e}_x$ and $\tilde{e}_y$ can be taken to be an arbitrary pair of orthogonal directions. The rotations can then be conveniently separated into the out-of-plane component and the in-plane component by executing Eq. 3b under the tangent coordinate system. Hence, the moment components of node $J$ and node $K$ ($\tilde{M}_J$ and $\tilde{M}_K$) under the tangent coordinate system can be expressed as

$$
\begin{align*}
\tilde{M}_J &= (M_J^\text{ext} - M_J^\text{int}) \\
\tilde{M}_K &= (M_K^\text{ext} - M_K^\text{int})
\end{align*}
$$

(6)

The two nodes are also merged in the calculations of the out-of-plane rotations but are detached in the calculations of the in-plane rotations. So the moment components become

$$
\begin{align*}
\tilde{M}_\text{merge}^\text{ext} &= (M_J^\text{ext} + M_K^\text{ext}) \\
\tilde{M}_\text{merge}^\text{int} &= (M_J^\text{int} + M_K^\text{int})
\end{align*}
$$

(7)

Likewise, the moment of inertia matrices contributed from beam element $i$ ($i = 1, 2, 3, 4$) also need to be transformed into the tangent coordinate system

$$
\tilde{I}_i' = \tilde{\Omega} \tilde{Q} \tilde{I}_i' \tilde{Q}^T
$$

(8)

where $\tilde{I}_i'$ is the moment of inertia matrix in the local coordinate system and $\tilde{Q}$ is the transformation matrix between local and global coordinate system. With the out-of-plane components being merged, the moment of inertia matrices of the scissor-like joint in and out of the tangent plane are

$$
\begin{align*}
\tilde{I}_\text{in} &= \tilde{I}_i' + \tilde{I}_i'' \\
\tilde{I}_\text{out} &= \tilde{I}_i'' + \tilde{I}_i'''
\end{align*}
$$

(9)

**Verification of the Beam Element**

The classic numerical test of a curved cantilever beam (Bathe and Bolourchi 1979; Moita and Crisfiea 1996) is employed herein to verify the formulation of the three-dimensional beam element when subjected to large deformation. As shown in Figure 4, the cantilever beam initially lies in the X-Y plane in the shape of an arch with a radius of 100 in (2540 mm) and a central angle of $\theta = 45^\circ$. The remaining material and geometrical properties of the cantilever are provided in Figure 4. With one end being fixed and the other end being applied with a vertical load $P=600$ lb (2667N), the cantilever undergoes large axial, bending and twisting deformations due to vertical loading and its curved geometry. The relation between the tip translations and the vertical force is shown in Figure 5 where $u$, $v$ and $w$ are the tip translations along the $x$, $y$ and $z$ directions respectively. It can be seen that the results from the proposed approach are in excellent agreement with those reported in previous studies (Bathe and Bolourchi 1979; Moita and Crisfiea 1996), demonstrating the validity of the formulation of the three-dimensional beam element. The cantilever was discretized into 40 equidistant particles to produce the numerical results in the present study.
FORM-FINDING EXAMPLE

In this Section, a form-finding example is presented to demonstrate the effectiveness of the proposed form-finding approach. In this example, the gridshell was assumed to be formed from pultruded GFRP tubes having an external diameter of 32mm and a wall thickness of 3mm, an elastic modulus $E = 30\text{GPa}$, a transverse shear modulus $G = 3\text{GPa}$ and a density $\rho = 1800\text{kg/m}^3$. The target surface employed in the example is a deep spherical dome (Figure 6a) with a radius $R = 3.25\text{m}$ and a height $h = 4.5\text{m}$. The initial planar grids were assumed to have a dimension of $12\text{m} \times 12\text{m}$ with a grid spacing of $0.5\text{m}$ (Figure 6b).

![Figure 6 A deep spherical dome imposed for gridshell form-finding](image)

Figure 6 A deep spherical dome imposed for gridshell form-finding

Figure 7 shows the final geometry of the gridshell resulted from Step 4 of the form-finding procedure. It should be noted that this final geometry is a very close approximation but not a precise reproduction of the target surface because a very small amount of snap-back of the grids inevitably occurs in Step 4 as a result of the removal of the rigid surface constraint and the pressure field. Figure 8 compares the final geometry of the gridshell with the geometry of the target surface. The comparison indicates that the difference between the two geometries is difficult to be spotted. The difference between the two geometries is further quantified by a careful examination of the distance between typical nodes (nodes 1 to 5 in Figure 8) on the final geometry of the gridshell and the center of the dome. The results of the examination are summarized in Table 1, which indicates that these nodes are located in the very near neighborhood of the target surface. Figure 9 finally shows the planar grids configuration in pursuit after releasing the pre-stresses in the deformed grids. The left and right plots in Figure 9 respectively shows the generated planar grids configuration and its position in the original planar grids.

![Figure 7 Final geometry of the gridshell](image)

Figure 7 Final geometry of the gridshell

![Figure 8 Comparison between final geometry of the gridshell and the target surface](image)

Figure 8 Comparison between final geometry of the gridshell and the target surface

<table>
<thead>
<tr>
<th>Node number</th>
<th>Distance (m)</th>
<th>Radius (m)</th>
<th>Error</th>
</tr>
</thead>
<tbody>
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<td></td>
<td>-0.031%</td>
</tr>
<tr>
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<td>0.123%</td>
</tr>
<tr>
<td>3</td>
<td>3.268</td>
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<td>0.554%</td>
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<td></td>
<td>0.831%</td>
</tr>
<tr>
<td>5</td>
<td>3.259</td>
<td></td>
<td>0.277%</td>
</tr>
</tbody>
</table>
7. Conclusions

This paper has been concerned with the development of a form-finding approach for bending-active FRP gridshells within an explicit dynamic framework. The continuous members are modelled using three-dimensional beam elements with a complete account for the nodal DoFs (six DoFs per node) so the bending and twisting behaviour of the members can be accurately modelled. The scissor-like joints are modelled using a pair of nodes whose nodal DoFs are all coupled except the in-plane rotation. It should be noted that the final geometry of the gridshell found by the proposed approach is a very close approximation but not a precise reproduction of the target surface because a very small amount of snap-back of the deformed grids inevitably occurs as a result of the removal of the rigid surface constraint and the pressure field in the proposed form-finding procedure.

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REFERENCES


EXPERIMENTAL ANALYSIS OF A REVERSE ELASTICA POP-UP GEOMETRY

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ABSTRACT
Pop-up mechanisms are able to generate complex forms from flexible planar materials. They have successfully been used for 3D micro scale engineering applications, but are not yet utilized for building-scale elements due to the difficulty of scaling planar actuation mechanisms. Separately, building elements formed with large, elastic bending deflections are widely used to for light weight and rapid assembly constructions. Commonly termed bending-active structures, their form utilises elastica geometries, which correspond to large bending deformations which have a minimum elastic potential energy. This paper will explore a novel actuation system for large scale pop-up structures that uses a reverse (pre-fabricated) elastica geometry. Experimental analysis is conducted on specimens made from fibre-reinforced-polymers (FRP), to compare elastic force-displacement behaviours of normal and reverse elastica samples under cyclic loading. A concept pop-up structure is also built to show the reverse elastica actuation system in practice.

KEYWORD
Pop-up, FRP, Bending-Active Structures, Elastica

INTRODUCTION
Pop-up mechanisms are those that utilize the kinematics of planar sheet folding to ‘pop-up’ a complex 3D shape. The mechanisms have parallels with origami and kirigami, which are the Japanese techniques for folding uncut and cut paper, respectively (Greenberg et al., 2011). Pop-up inspired design has wide potential for use in engineering applications, with several early uses already in existence. Novel deployable systems have been developed by embedding different complex motions and joints in planar origami states (Winder et al., 2009). Three-dimensional microstructures have been designed using layers of 2D planar materials, which are able to be assembled in a short duration, termed lamina emergent mechanisms (LEMs) (Gollnick et al., 2011; Gafford et al., 2013). The properties of LEMs have been studied and include influencing geometry, material properties, and boundary conditions. This knowledge has been used in recent designs to create self-actuating pop-up mechanisms (Jacobsen et al., 2010).

Self-actuating pop-up techniques, or self-pop-ups for short, have been widely applied on micro LEM devices. For example, a pop-up printed circuit microelectromechanical system was used to create a micro scale robot that can self-assemble and operate with simple design motions (Whitney et al., 2011). Self-pop-up through folding has also been achieved by applying global heating on 2D sheet materials to assemble a complex 3D cylindrical origami structure (Miyashita et al., 2013). Those studies have shown that self-pop-ups can successfully be applied on micro scale constructions. However, self-pop-up actuation systems are constrained at larger scale constructions due to the difficulty of scaling planar actuation mechanisms.

Bending-active structures are similar to pop-up mechanisms in that complex forms can be generated from planar materials. They utilize large elastic deformations to form a target profile, and in doing so utilise geometric non-linearities, such as stress-stiffening, to boost structural performance. Large-scale and lightweight constructions can therefore be effectively realized using a bending-active mechanism, if a material is selected which allows suitably large elastic deformations. Timber, fibre-reinforced polymer composites (FRPs), and hybrid materials are commonly used to due to their high flexural strength and low stiffness (Garti, 2015; Lienhard, 2014a).

The non-linear geometry of bending-active elements has led to some challenges in their modelling and design, however a range of techniques have been developed to address this. Particle-spring design methods were proposed for form-finding grid shells consisting of flexible members (Kuijvenhoven, 2012). The theory has contributed to digital form-finding tools such as the Kangaroo physics engine, which can simulate bending behaviors through dynamic relaxation of a spring-particle system (Symeonidou, 2015). Finite element structural analysis techniques have also been used to model flexible elements with uncertain boundary conditions, by using ultra-elastic contraction elements to actuate bending-active regions (Lienhard, 2014b). These simulation
techniques have been applied beyond individual elements to the design of full structures. For example, the Ongreening Pavilion uses a form-finding technique to generate a 3D grid shell composed of individual timber elements bent into an elastica curves (Harding et al., 2014). It is assembled in just 3 days and forms within a given inhabitable space 10m by 8m.

Elastica curves are a key feature of bending-active structures, as they correspond to the minimum bending energy configuration of a planar elastic shell. They have been investigated since the 13th century (Levien, 2008) and the mathematical approaches developed to model the form are still widely utilized today (Pacheco and Pina, 2007; Djondjorov et al., 2008). Elastica geometries are able to be obtained directly by using numerical solvers (Taki, 2007) to evaluate the fundamental equation, or they can be obtained using the same elastic large-displacement methods described above. Elastic bending can also be combined with other deformations to produce new forms of bending-active structures. Examples include spatial structures with twisted and interlaced bending elements (Nabaei et al., 2015); a deployable reflector made from a folded and bent composite curved-crease origami pattern (Pellegrino, 2006); and a pliable structural system inspired by parallel leaf-springs (Tornabell et al., 2014).

This paper investigates the intersection of pop-up mechanisms and bending-actives structures. Specifically, we propose a reverse (pre-fabricated) elastica element that can potentially act as a planar actuation system for large scale self-pop-up structures. First, the design and manufacture procedures for reverse elastica geometries will be developed. Second, an experimental analysis is conducted on normal and reverse elastica FRP samples to compare elastic force-displacement behaviours under cyclic loading. Third, the feasibility of a reverse elastica pop-up system is demonstrated with a concept pop-up structure.

GEOMETRY, MANUFACTURE AND TESTING

Geometry
There are numerous methods to model an elastica curve. This paper shall use the Kangaroo physics engine, which is freely available in the parametric CAD software Rhino/Grasshopper as follows. A straight beam element is specified with pinned-pinned support conditions and with a constant flexural stiffness along the element length. One support is moved in horizontal increments of 10% of the beam length, up to a total displacement of 100% of beam length. The Kangaroo engine simulates beam behaviour using dynamic relaxation of spring-particle system, with axial and rotational spring stiﬀnesses applied to sub-divided beam axes and joints, respectively. The relaxed beam geometry gives the resultant curves shown in Figure 1, and thus a numerical solution to the beam minimum energy configuration under elastic bending. Numerical profiles were validated with physical model data, with physical data obtained as the traced profile of an isotropic polycarbonate rectangular strip deformed in the same way as described for the above numerical simulation. A comparison of simulated and physical prototypes is shown in Figure 1 and it can be seen there is good correspondence.

Manufacture
A reverse elastica curve is one that has bending energy in a deployed state, i.e. it is fabricated in the deployed state such that when it returns to flat, it contains an elastic potential. It is unknown whether normal (straight-to-curved) and reverse (curved-to-straight) elastica beams have the same force-displacement behaviour, and so prototypes of each were manufactured and tested. For manufacture, prototypes were designed with 250mm length and 20mm width. It was calculated that a prototype thickness of approximately 1mm would allow the requisite large elastic displacements if constructed from FRP, and so each prototype was made with 5 layers of GFRP laminated with Ampreg 22 epoxy system. The first model, Type 1, was a flat strip. The second model, Type 2, was the reverse elastica curve and so was built with the profile shown in Figure 2, generated with the Kangaroo engine as describe above.
Testing
To assess the similarities and differences between normal and reverse elastica force-displacement behaviour, three samples of each type were subjected to a quasi-static cyclic displacement test with 10 cycles. Type 1 flat strips were actuated from a straight form into the design elastica curve, as shown in Figure 3(a). Type 2 reverse elastica curves were actuated from the design elastica curve into a straight form, as shown in Figure 3(b). For both, total actuation distance was 175mm. During the cyclic test, received load (N) in relation to the end point displacement (mm) was recorded at a 0.1s time interval. As the actuation between types is in reverse directions, displacement measurements were converted to an absolute percentage of total distance, with 0% representing the flat state and 100% representing the designed elastica curve state, as shown in Figure 4. The shape of the specimens was also recorded during testing with digital photographs.
RESULT AND DISCUSSION

Comparison of Manufactured and Design Geometry

The shape of the specimens under quasi-static cyclic loading was compared with the predicted elastica geometry. Type 1 models matched the elastica geometries simulated in Kangaroo physics throughout the entire process from loading to unloading, with the flat strip maintaining an elastica profile during deformation as shown in Figure 5. This suggests that the design composite material was able to achieve a large deformation with elastic bending behaviour. This conclusion is supported by the load-displacement histories shown in Figure 6a, which follow a reversible, and thus elastic, load path. The reverse elastica Type 2 similarly follows a reversible load path, as shown in Figure 6b, however Type 2 models were not able to return to a perfectly flat state with a 175mm displacement. This can be understood with consideration of averaged force-displacement responses.

Figure 5 Specimens maintained elastica forms under quasi-state cyclic loading (green curves are the simulated elastica profiles).

Figure 6 Experimental data. (a) Type 1 normal strip. (b) Type 2 reverse strip.

Comparison of Normal and Reverse Elastica Geometries

Figure 6 above shows the complete force-displacement time histories for all tested samples. It can be seen that there is good load path reversal and it can be concluded that all samples were loaded in the elastic region throughout testing. Figure 7 below shows the average force-displacement relationship for each type, obtained by averaging the 10 cycles for each sample and then averaging across samples. Type 1 specimens show an initial linear elastic region up to some critical buckling load, after which the response is softened. This is classic non-linear elastic buckling behaviour. Type 2 models, starting at 100%, begin with a softened response and then stiffen as the samples approach the straight, 0% configuration. The magnitude of force change between end states is similar between types, however the path and thus stored energy is different, particularly in the near-
straight region. This is because the Type 2 models start as a post-buckling form and so do not experience the linear elastic region and subsequent bifurcation seen in Type 1 models.

![Figure 7](image.png)

**Figure 7** The average result of a single cycle for both models under quasi-state cyclic loading.

**Self-pop-up Actuation System**

Reverse elastica geometries can be used to develop 3D bending-active structures that store bending energy in a flat state, rather than bent state. Thus, they can be used for structures that are stored in a planar state but can pop-up to a bent configuration, i.e. they can be used as an actuation system for self-pop-ups. A design which utilises this is proposed in Figure 8. Eight reverse elastica strips were fabricated and joined together with hinges at one end of the strips. These strips can be flattened and held back-to-back with clamps, as shown in Figure 8(a). As elastica geometries are minimum energy forms, external energy required to flatten the structure is minimal. Once clamps are removed the lock positions in flat-storage, the flat strips pop-up transform into the reverse elastica form, as shown in Figure 8(b). These can be arranged as shown in Figure 8(c) to form a stable deployed structure.

![Figure 8](image.png)

**Figure 8** Self-pop-up reverse elastica structure. (a) Flat-storage state. (b) Self-actuated state. (c) Final form.

**CONCLUSIONS**
Pop-up techniques are currently constrained at large scales by the lack of macro-sized, flat-packable actuation systems. This paper has explored an actuation system that is potentially capable of addressing this lack by connecting elastica geometries, pop-up mechanisms, and large-scale bending-actives structures. Reverse elastica geometries were produced using a dynamic relaxation analysis and were manufactured from a GFRP composite. Cyclic testing showed both models exhibited the predicted elastic large-displacement behaviour. Flat-to-curved samples showed a classic non-linear elastic buckling behaviour and curved-to-flat reverse elastica samples only exhibiting a non-linear post-buckling behaviour, i.e. there was no return to an elastic region and no bifurcation. Reverse elastica mechanisms were also able to self-pop-up a simple concept structure.

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REFERENCES


Special Session on Natural Fibre Composites in Construction

Organizers:
Guijun XIAN
Libo YAN
NATURAL MATERIAL COMPOSITES FOR EARTHQUAKE-RESISTANT STRUCTURES

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ABSTRACT

Natural materials such as coconut fibre or flax fabric, are environmentally friendly and can also be used as reinforcement in polymer-concrete composites. Coconut fibre in concrete will not only enhance the ductility of the structural members due to a fibre bridging effect, but it will also increase the material damping due to friction at the interface of widely distributed small cracks. Flax fabric can be used to significantly enhance the tensile property of flax fabric reinforced polymer (FFRP) composite. A combination of FFRP and coconut fibre reinforced concrete (CFRC) as construction materials thus has the merits of the two composites. For earthquake-resistant structures these composite materials do not have the disadvantage of conventional steel-reinforced concrete, because steel corrosion will no longer be an issue. In addition, the structure is lighter. Consequently, FFRP-CFRC structures will experience less impact of earthquake loading because of the lower inertial forces activated in the structure. The natural based structures can also subscribe to a low-damage design philosophy in which the structural members are allowed to partially separate while responding to earthquake loading. An overview of current research on innovative seismic design approaches using natural material composites is presented.

KEYWORDS

Low-damage seismic design, flax FRP, coconut fibre, earthquake-resistant structures.

INTRODUCTION

Current seismic design philosophy tolerates a degree of damage at locations predefined by the designer as long as the safety of inhabitants is assured. Current design specifications reduce construction costs by tolerating plastic deformations at predefined locations (called plastic hinges). Observations of damage to structures following the Canterbury earthquakes (Chouw and Hao, 2012), confirmed that the structures designed to seismic design standards will behave as the designer intended.

Figure 1 shows two adjacent buildings minimal visible damage from the outside. However, inside the buildings RC beam-column joints suffered damage as the designer intended, while the occupants remained safe. However, repair costs are often high (90% of the new building value). In addition, costs accrue because structures and infrastructure are no longer fit for purpose (down time). These costs are considerably difficult to predict.
To avoid these unpredictable costs due to down time of the structures which can easily paralyze the whole city, a low-damage philosophy has been proposed. One possible way to achieve low-damage earthquake-resistant structures is to permit partial separation (uplift) of the footing of the structure, while responding to earthquake loading. Consequently, the structure itself will also temporarily separate from the ground. This separation can lead to significantly less damage, and this has been confirmed by observations after major earthquakes.

The first bridge with the capability to separate at its footings was the South Rangitikei Viaduct, built in the North Island of New Zealand (Beck and Skinner, 1973). The bridge has the ability to perform out-of-plane rocking. Because of the separation at the footings before elastic damage is incurred, rigid-body movements of the bridge structures are activated. Consequently, little or no local deformation along the bridge piers occurs. The impact of the earthquake and the corresponding resulting damage is thus significantly reduce or even entirely avoided.

Chen et al. (2016) presented the most recent research outcomes. The authors focused on the impact of near-source earthquakes on upliftable bridges. This research is especially important because near the earthquake source, the vertical component of the ground surface motions can be much stronger than those components in the horizontal directions. The influence of a simultaneous excitation of high-frequency and strong vertical component and horizontal components has hitherto seen little investigation.

Another example of an upliftable structure is the chimney near the Air New Zealand hanger at Christchurch airport (Figure 2). The chimney can separate at the base when it experiences ground motions larger than a predefined threshold. Despite a number of large earthquakes and more than 10,000 aftershocks following the Canterbury earthquakes, including strong aftershocks such as those on the 22 February 2011 and the 25 May 2012, no structural damage has been observed by the author in their field investigations (Chouw and McCue (2012)).

The beneficial effect of temporary and partial separation at the structural support has been confirmed by Qin et al. (2013) in their physical experiments using a shake table. The consequence of footing separation and subsoil for the structural response and for the response of secondary structures were investigated.

Loo et al. (2012, 2014, 2015 and 2016) focused on the seismic performance of upliftable timber walls. To enhance the overall performance, slip-friction connectors are installed at the bottom corners of the wall. To resist the shear force at the base shear keys are implemented around the middle of the wall base. The frictional sliding activated during the wall loading is used to dissipate the energy induced into the wall from an earthquake. Additionally, the slip-friction connectors permit vertical uplift of the wall, and rigid body movement of the wall occurs during this uplift phase. With the slip-threshold of the connectors appropriately defined damage to the wall is thus minimized or avoided altogether. Timber walls with slip-friction connectors thus compare favourably with conventional shear walls, in which the ductility is activated in the form of plastic behaviour, and hence damage to the nail connections.

Ormeño et al. (2015a and b) investigated the consequence of uplift on the interaction between fluid, storage tank and the supporting soil, during strong earthquakes. The temporary separation at the base of the tank will not only activate rigid-body movements of the wall, but also alter the interaction between the fluid and the tank wall. To enhance the damping behaviour of the whole fluid-structure-soil system slip-friction connectors at the base of the tank were considered. The shake table experiments have confirmed the beneficial influence of uplift on the overall seismic performance of the fluid-structure-soil system, while the slip-friction connectors can be used to control the uplift movements.
NATURAL FIBRE REINFORCED POLYMER-CONCRETE COMPOSITES AS EARTHQUAKE-RESISTANT CONSTRUCTION MATERIALS

Static response

To incorporate natural materials as possible construction materials of future earthquake-resistant structures, the University of Auckland Centre for Earthquake Engineering Research (UACEER) has developed a composite of flax fabric reinforced polymer (FFRP) and coconut fibre reinforced concrete (CFRC). Yan and Chouw (2013a and b, 2014) and Yan et al. (2014, 2015a and b) have performed a number of experiments on FFRP-CFRC composite members. In addition to static and dynamic properties of the structural members, the influence of the surroundings, for example UV and sea water, and the durability of the composite have been investigated.

To reduce the impact of dynamic loadings, Chen and Chouw (2015, 2016a and b) proposed the usage of double FFRP confinements of the FFRP-CFRC composite bridge pier. Because of the inner confinement, the amount of CFRC is reduced and the bridge pier thus has less mass. This results in less activated inertia force. Consequently, the bridge pier experiences less impact from the dynamic load.

Figure 3 shows the influence of coconut fibre, flax fibre and double FFRP confinement on the load bearing capacity of a beam. With coconut fibre the mid-span deflection can be increased by almost 100% (compare the dotted line with the bold dashed line in Figure 3(b)). With FFRP confinement the load bearing capacity can be increased by more than 700%. An additional coconut fibre further increases the load bearing capacity and the ductility of the structure. The consequence of inner confinement of the structure for the load bearing capacity can be seen in Figure 3(c). When reaching certain thresholds, slippages take place at the interface between outer FFRP layer and the concrete, but with double confinement the slippage can be entirely avoided (see dotted line in comparison with the solid line).

Dynamic response

To investigate the seismic response of the FFRP-CFRC bridge pier, the ground motion is simulated based upon the Japanese design spectrum for hard soil condition (JSCE, 2000). Figure 4 shows the design spectrum and the response spectrum of the simulated ground motion with similar frequency content.

Figure 3 Bending test. (a) Set-up, influence of (a) FFRP and CFRC and (b) double FFRP confinement

Figure 4 Japanese design spectrum and response spectrum of the ground acceleration
Despite less concrete being used in the bridge pier with double FFRP confinement, Figure 5 clearly shows that the bridge pier with single (FFRP, solid line) and double (DFFRP, dashed-dotted line) confinement perform very similarly. The periods of the responses show hardly any difference. The result indicates that the inner FFRP confinement can compensate for the reduced concrete component without causing too much change in the dynamic properties of FFRP-CRFC and DFFRP-CFRC composite piers.

**Fibre reinforced lightweight concrete**

To further reduce the impact of dynamic loadings on a structure, lightweight volcanic pumice rocks have been considered as both coarse and fine aggregates (see e.g. Anwar Hossain, 2004, Hassanpour et al., 2012, and Hossain et al., 2011). Figure 7 shows the Opak Fault south of Yogyakarta City in Indonesia. The white marks indicate the pumice rocks observed by the author during his field investigation. Figure 8 shows the behaviour of the lightweight concrete. The pumice aggregates can be clearly seen in Figure 8(a). While the concrete without fibre reinforcement simply splits without resistance, coconut fibres clearly hold the concrete together (Figure 8(b)).
LOW-DAMAGE EARTHQUAKE-RESISTANT STRUCTURES USING NATURAL FIBRE REINFORCEMENT

Partial and temporary separation throughout the structure

To reduce the impact of earthquake loading, interlocking blocks have been developed by UACEER (Ali et al., 2012, 2013a and b, Tang et al., 2014). Coconut fibre is used to enhance the performance of the blocks. Whenever the blocks experience excitations that are stronger than they are designed to endure, the blocks separate from adjacent blocks. Consequently, rigid block movements are activated and the impact of the loading is significantly reduced. Figure 6(a) shows one of the developed block. The holes are for the coconut ropes that limit uplift movement so that dislocations of blocks will not take place. Figures 6(b) and 6(c) show the interlocking block column and wall, respectively. Despite the structures having experienced a number of severe earthquake loadings no damage can be observed.

CONCLUSIONS

A brief overview of research activities at the University of Auckland Centre for Earthquake Engineering Research has been provided. The goal is to develop eco-friendly construction materials that can be used in future low-damage earthquake-resistant structures. To reduce the impact of earthquakes, temporary and partial separation at the base or throughout the structure is permitted. The separation activates rigid body movements of the structural members and thus can eliminate the local deformation of the structure. Reduced or even zero local deformation minimises or completely eliminates excessive stresses building up. Consequently, damage to the structure is minimized. The natural materials considered are flax and coconut fibre and pumice aggregates and these are demonstrated to have significant promise as a construction material in seismic resistant structures.
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USING NATURAL FIBRES AND NATURAL FIBRE REINFORCED POLYMER COMPOSITES AS REINFORCEMENT MATERIALS OF CONSTRUCTION STRUCTURES

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ABSTRACT

Natural fibres and their fibre reinforced polymer (FRP) composites are light-weight and environmentally-friendly materials which can be used as sustainable construction and building materials. Among different natural fibres, coir has the largest toughness which can be used as reinforcement within concrete (i.e. coir fibre reinforced concrete (CFRC)) to increase concrete mechanical properties. Flax has favorable specific tensile properties which are comparable to those of synthetic E-glass fibres, thus flax fibre reinforced polymer composite (FFRP) can be used in the similar ways of glass FRP as reinforcement of different construction structures. A combination of prefabricated FFRP tube and a CFRC core can be formed an innovative steel-free FFRP-CFRC (i.e. FFRP tube encased CFRC) hybrid structure possessing the advantageous of both components. Current study provides an overview of (1) structural performance of FFRP-CFRC hybrid structures under static and dynamic loadings and (2) use of FFRP as external strengthening material of concrete and timber structures.

KEYWORDS

Flax FRP, coir fibre reinforced concrete, timber, confinement, strengthening

INTRODUCTION

Construction industry is the major and most active sector in the worldwide which accounts for the consumption of large amounts of non-renewable resources and the depletion of large amounts of greenhouse gas emission leading to severe environmental issues (Torgal and Jalali 2011). The severe environmental issues calls for the development of sustainable concrete and structures urgently (Yan and Chouw 2012, Yan 2012). In the past decades, bio-fibre reinforced concrete has been widely investigated because the short fibres were able to improve tensile and flexural strength, toughness and impact resistance when being used within cementitious (Yan and Chouw 2013a,b). Among various fibres, coir, because of its highest toughness and the extremely low cost and availability, is a good fibre reinforcement for concrete material, i.e. coir fibre reinforced concrete, termed as CFRC (Yan et al. 2016a). Recently, utilizing bio-fibres from natural resources to replace synthetic fibres (e.g. carbon/glass) for FRP composites application has gained popularity (Yan et al. 2013a, 2013b, 2013e) due to their economic, environmental and social significances (Yan and Chouw 2014a,b). Among different natural fibres, flax provides the best potential combination of low cost, light weight, and high strength and stiffness for structural applications (Yan and Chouw 2015). The specific mechanical properties of flax fibres are comparable and even superior to those of E-glass fibres (Yan et al. 2015a, Yan et al. 2016). Thus, it is realistic to use flax FRP composites for infrastructure application in the ways similar to the glass FRP composites. For example, flax FRP can be used as external strengthening materials of different structures such as concrete and timber and used in innovative structural systems in the configuration of prefabricated tube, i.e., concrete filled FRP tube (CFFT). A combination of prefabricated flax fabric reinforced polymer composite (FFRP) tube and the CFRC core formed a new representative of CFFT (i.e. FFRP tube encased CFRC, FFRP-CFRC) possessing the merits of both components (Yan and Chouw 2014c). Thus, current study provides an overview on the structural performance of FFRP-CFRC structures under static and dynamic loadings, and the use of FFRP as external strengthening materials of concrete and timber structures.
NATURAL FLAX FIBRE REINFORCED POLYMER COMPOSITES USED IN INNOVATIVE HYBRID STRUCTURE

Figure 1 shows a FFRP-CFRC composite structure subjected to axial compression and four-point bending tests. In the FFRP-CFRC, the pre-fabricated tube made of FFRP acts as permanent formwork for fresh concrete and also provides confinement to concrete. In addition, as a protective shell, the tube can prevent the inner CFRC from outer aggressive environmental attacks. Short monofilament coir fibres are the reinforcement within the concrete core to reduce crack width and amounts and in turn modifies the failure mode of the concrete structure from brittle to ductile as a result of fibre bridging effect, see Figure 1(c) (Yan et al 2014c; Yan et al 2015b).

Figure 1 FFRP-CFRC composite specimens subjected to axial compression (a) and four point bending tests (b) and fibre bridge effect (c)

The axial compressive stress-strain curves of plain concrete (PC) and 4-layer FFRP tube confined plain concrete (FFRP-PC), CFRC and 4-layer FFRP-CFRC are displayed in Figure 2. It is clear that the axial stress-strain responses of both FFRP-PC and FFRP-CFRC specimens show a distinct bi-linear manner, namely, an initial linear response and a second ascending linear response connected by a transition zone (Yan and Chouw 2013a,b). When the applied axial stress is low, the stress-strain response of FFRP-PC or FFRP-CFRC is similar to the unconfined PC or CFRC. When the applied stress approaches the peak strength of the unconfined PC or CFRC, the curve enters a short nonlinear transition region where considerable micro-cracks are propagated and the lateral expansion significantly increased in the concrete. The second linear curve of FFRP-PC or FFRP-CFRC displays an ascending branch, indicating the good confinement of the FFRP tube. The linear response in this region is mainly dominated by the structural behaviour of FFRP composites (Yan and Chouw 2013d, 2013e). The compressive strength of unconfined PC and CFRC is 25.8 MPa and 28.2 MPa and the ultimate axial strain at the peak strength is 0.20% and 0.54%, respectively. It shows that the addition of coir fibre increases the compressive strength and the axial strain because of the fibre bridging effect. The ultimate compressive strength of FFRP-PC and FFRP-CFRC is 53.7 MPa and 56.2 MPa, respectively, corresponding to the confinement effectiveness of 2.08 and 2.0 for FFRP-PC and FFRP-CFRC with the tube thickness of 4 layer flax fabric. The data here indicates that the FFRP tube confinement increases the load carrying capacity of PC and CFRC up to 108% and 100%, respectively. The ultimate axial strain of FFRP-PC and FFRP-CFRC is 2.25% and 2.70%, respectively. So, compared with the ultimate axial strain of PC, the axial strain ratio for FFRP-PC and FFRP-CFRC is 11.3 and 13.5, respectively, indicating that the FFRP tube confinement increases the ductility of PC and CFRC remarkably. Therefore, the FFRP tube confinement increases the load carrying capacity and ductility of concrete significantly.

Figure 2 Axial compressive stress-strain behaviour of FFRP-PC and FFRP-CFRC

The transverse load versus mid-span deflection responses of PC, FFRP-PC, CFRC and FFRP-CFRC under four-point bending test are displayed in Figure 3. The PC specimen exhibited a linear response up to failure showing a pure brittle failure of the concrete. The curve was very short indicating the negligible load carrying capacity and deflection of the specimen. For CFRC, the curve also shows a linear response up to the peak load but it followed by a post-softening response, indicating a ductile behaviour of the specimen. It is believed that the coir fiber inclusion is responsible for this post-peak response. For FFRP tube confined concrete specimens, it can be seen
that the response of the confined concrete specimen is dominated by the strength and stiffness of the FFRP tube. Both the FFRP tube confined PC and CFRC specimens possessed a nonlinear load-deflection response before approaching the peak load. Like that of unconfined PC, the FFRP-PC specimen also exhibited a brittle failure as the result of the non-yielding characteristics of FFRP materials. However, on the other hand, the FFRP-CFRC specimen had a post-softening curve with a ductile response, showing the advantageous of coir fibers. For the confined PC and CFRC, it is clear that there were some sudden load drops in the load-deflection curves, which can be explained by the occurrence of the slippage between the concrete core and the FFRP tube. Slippage between the concrete core and the FFRP tube may compromise the ultimate load carrying capacity of the FFRP and concrete composite column. Therefore, special arrangement should be considered to roughen the inner surface of the FFRP tubes to prevent slippage and may further increase the load carrying capacity of the composite column (Yan and Chouw 2013a).

In general, FFRP-CFRC could provide a good alternative to conventional reinforced concrete (RC) in corrosive environments, e.g. highway bridge piers and girders, marine fender piles, poles and overhead sign structures. These structures are periodically subjected to various dynamic actions from heavy vehicles, wind, ocean waves and earthquakes (Yan et al. 2014b, 2014c). The periodic response of a bridge component to, e.g. wind loading, may lead to material fatigue and thus raise safety concerns. A good understanding the dynamic properties of FRP confined concrete structures, like damping and natural frequencies has industrial significance (Yan and Chouw 2014c). Thus, the PC, CFRC, 4-layer FFRP-PC and 4-layer FFRP-CFRC composite beams subjected to hammer-induced vibration tests were considered. Long cylindrical beams were tested to determine the fundamental frequencies of the transversal, longitudinal and torsional vibrations for calculating the dynamic modulus of elasticity and Poisson’s ratio followed ASTM C215 and for determining the damping ratio (Yan and Chouw 2014c). The tested results indicated that with the addition of coir fibre to CFRC, the damping ratio, in the transversal, longitudinal and torsional vibration modes, increases by 362% (from 0.79% to 3.65%), 301% (from 0.93% to 3.73%) and 280% (from 0.86% to 3.27%), respectively, compared with the PC. This data indicates that coir fibre inclusion has a significant influence on the damping of the CFRC composite (Yan and Chouw 2014c). Test results also showed a similar increase pattern of both FFRP-PC and FFRP-CFRC at all the three vibration modes. With an increase in tube thickness from 2-layer to 4-layer flax fabrics, the damping ratios of both FFRP confined PC and CFRC increase. In CFRC, the coir fibre inclusion produces more interfaces and stress transition zones in the cementitious matrix. During vibrations, more energy is dissipated due to the internal friction between the coir fibres and the matrix where more fibre/cementitious matrix interfaces are involved. In addition, concrete is a brittle material with extensive potential micro-cracks and these cracks may open and close during vibration, and the matrix interacts with the fibre surface, resulting in energy loss (Yan et al. 2016b). For FFRP confined concrete, FFRP tube introduces new interfaces between the tube and the concrete core, which may also be responsible for dissipating energy by friction during the vibration, thereby increasing the damping ratio of FFRP confined concrete. Therefore, both coir fibre and FFRP tube improve the damping ratio of the FFRP tube confined CFRC, thus reducing the effect of dynamic loading on the structure.

Due to the excellent dynamic properties obtained from vibration tests, it is expected that the FFRP-CFRC has potential to be seismic-resistant structure, thus, the seismic performance of FFRP-CFRC columns as bridge pier subjected to earthquake loadings was considered using a shake table (Figure 4). Earthquake loadings based on Japanese Design Spectrum for medium soil were considered. Four different sets of earthquake loading (i.e. set 1, 6, 12 and 18) with three different scale factors (i.e. 0.5, 0.8 and 1.0) were applied. Figure 5(a) shows the relative top displacement of FFRP-CFRC columns under one of the simulated ground motions and Figure 5(b) shows the summarized maximum top relative displacement for all the load cases. It can be seen that the maximum top relative displacement in set 6 is even larger than the 5% allowable drift for seismic design of building frames in most design codes, indicating the potential of this FFRP-CFRC as seismic-resistant structure (Yan et al 2014d).
Figure 4 Shake table test of FFRP-CFRC column

Figure 5 The relative top displacement – time history under one of the simulated ground motions (a) and the maximum relative top displacement of FFRP-CFRC column in all load cases (b)

NATURAL FLAX FIBRE REINFORCED POLYMER COMPOSITES AS EXTERNAL STRENGTHENING MATERIALS

For timber structure

Another use of natural FFRP composite as reinforcement is to be external strengthening materials of conventional construction and building materials. For example, FFRP composites in the configuration of = plates can be used external reinforcement of timber structure placed at the bottom section of the wood beams. Figure 6(a) shows the failure modes of 1-, 2- and 3-layer flax FRP plate strengthened wood beams (Douglas fir) under four-point bending and Figure 6(b) shows the micrograph of the flax FRP and wood interface using the Scanning Electronic Microscope (SEM) study. It is clear that all the wood beams strengthened with 1-, 2- and 3-layer FFRP failed with the rupture of the FFRP and wood beam at the middle span. No delamination or debonding failure was observed for tested beams, indicating the good compatibility of FFRP and wood beams. The SEM study also indicates the good interfacial bond between the FFRP and the wood beams. The flexural test results showed that the average peak lateral load of the un-strengthened wood beam obtained from three identical specimens was 3.3 kN, while the value of 1-, 2-, and 3-layer flax FRP strengthened beams was 4.5 kN, 5.5 kN and 6.2 kN, respectively. For the energy under the area of the load-deflection curves, that of un-strengthened wood beam was 20.5 J and the energy of 1-, 2- and 3-layer FFRP strengthened beams was 40.0 J, 67.5 J and 104.4 J, corresponding to an increase of 94.9%, 229.2% and 409.0%, respectively. The FFRP external reinforcement also enhanced the ultimate deflection of the beams significantly, with an increase of 36.2%, 32.1% and 76.2% respectively for the beams with 1-, 2- and 3-layer FFRP strengthening. However, it should be pointed out that the use of FFRP strengthening does not change the failure pattern of the wood, which is still a brittle failure.
For concrete structure

Natural FFRP composites can also be used as external strengthening materials of concrete structures (Huang et al. 2016). Figure 7 shows the relationship between the ultimate compressive strength of the plain concrete confined by FFRP wrappings obtained from uni-axial compression. As can be seen, the increase in ultimate compressive strength is almost proportional to an increase in the number of FFRP wrappings. For the unconfined PC, the average ultimate stress was 20 MPa, while that of the 2-, 4-, and 6-layer FFRP wrapping confined plain concrete was 28 MPa, 37 MPa and 49 MPa, respectively (Yan 2016). Thus, FFRP wrapping is an effective external strengthening material to improve the load carrying capacity of concrete column.

Figure 8 shows the load-deflection responses of PC and CFRC beams strengthened without and with 6-layer FFRP composite plates under three-point (3PB) and four-point (4PB) bending tests. It is clear that the use of FFRP strengthening increased the peak load, deflection and the energy capability of both PC and CFRC beams significantly, i.e. under 4PB, the peak load of 6-layer FFRP strengthened CFRC was 31 kN while that of CFRC without strengthening was only 6.2 kN. Furthermore, the use of FFRP strengthening changed the post-peak response of both PC and CFRC beams. For both PC and CFRC beams without FFRP strengthening, the failure was brittle while the strengthened beams showed somehow a ductile behavior (Yan et al 2015b).
CONCLUSIONS

This study provided a brief overview of using natural flax fibre reinforced polymer composites used in innovative hybrid structure and used as external strengthening materials of concrete and timber structures. In general, the FFRP-CFRP hybrid structures showed their potential to be axial, flexural and seismic-resistant structural members. As external strengthening materials, the use of FFRP in different configurations can improved the ultimate load carrying capacity, deformation and energy dissipation capacity of concrete and timber structures effectively.

REFERENCES


FLEXURAL BEHAVIOR OF RC BEAMS STRENGTHENED BY NATURAL FLAX FRP PLATES

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ABSTRACT
This paper reports an experimental investigation on flexural strengthening of RC beams with natural flax fabric reinforced polymer (FFRP) composite plates. The experimental variables considered are FRP thickness (4 and 6 layer), the internal steel rebar ratio (0.223% and 0.503%) and the pre-cracking of RC beams (i.e. by applied 80% yielding load of control RC beam before the application of FFRP plates). This study shows that FFRP plates strengthening increased load, deflection and ductility of RC beams remarkably. The enhancement in load and ductility is more pronounced for RC beams with lower steel ratio. All the FFRP strengthened RC beams exhibited a similar failure pattern, namely, initial steel yielding and then rupture of the FRP plates. There was no obvious effect of pre-cracking on the load of the FFRP strengthened RC beams, showing that the effectiveness of FFRP external strengthening materials for retrofitting damaged RC beams (e.g. after earthquake-attack). In addition, the experimental results are compared with the predictions based on the equations given in ACI 440.2R-08 for beams strengthened with synthetic FRP composites.

KEYWORDS
Fabrics, delamination, mechanical properties, FRP composites.

INTRODUCTION
Nowadays, because of the increased environmental concern, there is an increasing tendency to use natural fibres to replace synthetic glass or carbon fibres as reinforcement materials of FRP composites for civil engineering applications (Dittenber and GangaRao 2012; Yan and Chouw 2013a; Yan et al. 2014a; Cevallos et al. 2015). The advantages of natural fibres are the dependence on non-renewable energy/material sources, lower pollutant emissions, lower cost and greenhouse gas emissions, enhanced energy recovery and biodegradability (Yan and Chouw 2013b). Among natural fibres, flax is widely used fibre reinforcement for FRP because it offers the best potential combination of low cost, light weight, and high strength and stiffness for structural applications (Yan et al. 2014b). Thus, a series of studies were conducted on the structural performance of natural flax FRP composite (FFRP) tube encased coir fibre reinforced concrete hybrid structures (Yan and Chouw 2013b, 2013c, 2013d; Yan et al. 2014c). Previous studies (Yan and Chouw 2013b, 2013c, 2013d; Yan and Chouw 2014; Yan et al. 2014c) showed a good potential of FFRP tube confined concrete as axial, flexural and impact-resistant structural members. The composite structure also showed better structural performance (i.e. energy dissipation and load carrying capabilities) than conventional steel reinforced concrete (Yan and Chouw 2013d). To date, few studies have been considered using FFRP plates as external strengthening materials of RC members. Whether the environmentally-friendly and cheap FFRP composite is effectively for strengthening of existing deficiently-designed RC structures and/or for retrofitting of damaged RC structures after earthquake-attack is a matter of interest. Therefore, this study investigates the flexural performance of RC beams strengthened with externally bonded FFRP plates.

EXPERIMENTAL SECTION
In this study, eight RC beams were constructed. All the beams had a rectangular cross section, with a length of 2000 mm, width of 150 mm and depth of 300 mm. For the beams with FFRP strengthening, three of beams were strengthened with 4 layers of FFRP and the other three beams were strengthened with 6 layers of FFRP. To evaluate the effect of steel reinforcement ratio, two types of steel reinforcement ratio were used: 0.223% and
0.503%, respectively. In addition, two RC beams (FB-5 and FB-6) were pre-cracked by applying 80% of the yield load (this load level can lead to a few small cracks of RC beams but do not cause total loss of load carrying capacity of the RC beams) of the control beam. After pre-cracking, the two RC beams with micro-cracks were then strengthened with FFRP laminates so as to investigate the feasibility of FFRP as external strengthening used to retrofit damaged RC structures after earthquakes.

For dimension of the RC beams is 2000 mm in length, 150 mm in width and 300 mm in height. The length of FFRP sheets used is 1700 mm and the width of the FFRP sheet is 140 mm. The FFRP sheets were glued at the bottom of the pre-treated RC beams with the epoxy resin adhesive. To have the FFRP with a proper workability, three U-shaped FFRP bands were added at both ends of the beam to prevent the main FFRP plate at the bottom of the beam from debonding. The U-shaped FFRP bands (60 mm in width and 700 mm in length) had one layer of FFRP and spaced at 30 mm.

The clear span of the specimens was 1800 mm and the span of the beam between the two loading points was 600 mm. The beams were loaded in four-point bending and tested until failure with a hydraulic actuator. It should be noted that the hydraulic actuator applied concentrated loading on the midpoint of a rigid steel spreader beam. In order to obtain the FRP strain at mid-span, nine strain gauges were stalled on the surface of FRP sheet, which were connected to a data acquisition system to monitor the strain during loading. Moreover, five displacement transducers were used to measure deflection at five different locations in each span, at mid-point, two loading points, and both ends of the beam.

RESULTS AND DIACUSSION

In general, the failure of the tested beams can be divided into two modes. The control beams (CB-1 and CB-2) failed with steel yielding, followed by concrete crushing (Mode I). While the FFRP reinforced beams (FB-1–FB-6) failed with steel yielding, followed by FFRP rupture (Mode II). The capacity of crack load, yield load, ultimate load, deflection at yield load, deflection at ultimate load, deflection at failure, failure mode, ductility index and the energy absorption capacities are listed in Table 1.

<table>
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<th>Specimen</th>
<th>Steel</th>
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<th>( P_c ) (kN)</th>
<th>( P_y ) (kN)</th>
<th>( P_u ) (kN)</th>
<th>Defle. at ( P_y ) (mm)</th>
<th>Defle. at ( P_u ) (mm)</th>
<th>Defle. at failure (mm)</th>
<th>Failure mode</th>
<th>( \mu_{\Delta y} )</th>
<th>Energy absorption (kN·mm)</th>
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<td>11.0</td>
<td>43.7</td>
<td>79.0</td>
<td>2.4</td>
<td>16.9</td>
<td>17.90</td>
<td>II</td>
<td>7.1</td>
<td>1094.8</td>
</tr>
<tr>
<td>FB-6</td>
<td>2 C 8</td>
<td>6</td>
<td>15.0</td>
<td>53.0</td>
<td>98.3</td>
<td>3.3</td>
<td>22.8</td>
<td>23.89</td>
<td>II</td>
<td>7.0</td>
<td>1821.8</td>
</tr>
</tbody>
</table>

Load-Deflection Behaviour and Load-FFRP Strain Relationships

From Figure 1 (a) and Table 1, it is clear that the FFRP reinforcement increases the ultimate load capacity of all the strengthened beams in varying degrees. The specimen FB-1 with 4-layers FFRP and steel reinforcement ratio of 0.223%, compared to the specimen CB-1 with the same steel reinforcement ratio, the ultimate load increases 67.7% from 46.3 kN to 77.7 kN, while the specimen FB-2 with 6-layers FFRP and steel reinforcement ratio of 0.503%, compared to the specimen CB-1 without FFRP but with same steel ratio, it increases 105.1% from 46.3 kN to 95.0 kN. It can be concluded that more layers of FFRP strengthening lead to higher ultimate load capacity of the beams, when comparing the values of these 3 groups: FB-1 and FB-2, FB-3 and FB-4, FB-5 and FB-6. In addition, the increase in ultimate load of specimen FB-3 and FB-4, compared to the specimen CB-2 without FFRP, are 15.5% (from 142.3 kN to 164.3 kN) and 21.1% (from142.3 kN to 172.3 kN), respectively. It should be noted that the steel reinforcement ratio of specimens FB-3 and FB-4 (0.223%) is lower than FB-1 and FB-2 (0.503%). This data here indicates that the increase in the ultimate load capacity for RC beam with low-reinforcement ratio due to FFRP strengthening is larger than that of beam with high-reinforcement ratio. Additionally, it can be seen that the pre-cracking has no obvious effect on the ultimate load carrying capacity of FFRP strengthened beams.
For the deflection behavior, the test results show that specimen FB-2 with 6 layers FFRP has the largest deflection among all specimens. Compared to the control specimen CB-1, the deflection at failure of specimen FB-1 and FB-2 increases 92.3% and 147.4%, respectively. The deflection at failure of FB-3 (22.1 mm) and FB-4 (22.3 mm) are 16.5% and 17.6%, respectively, larger than that of the control specimen CB-2 (19.0 mm). Moreover, the deflection at failure of FB-5 (17.9 mm) and FB-6 (23.9 mm) are slightly higher than of the corresponding deflection of FB-1 and FB-2, respectively. From Figure 1 (a), it is obvious that all FFRP reinforced specimens show a brittle failure in post-peak response with a sharp drop in load with the deflection. This implies the failure of FFRP strengthened beam is determined by the brittle characteristics of FFRP laminates. It also should be pointed out that the same increase in load increment contributes to a larger growth in deflection of the strengthened beams after the yield load.

Figure 1 (b) shows the load-FRP strain relationship at the mid-span of all 6 FFRP reinforced specimens. Compared to FB-1, the strain for FB-2 specimen increases only 2.3%. FB-4 increases 52.0% compared to the strain of FB-3. FB-6 increases 77.8% compared to that of FB-5. This indicates that more layers of FFRP strengthening correspond to a larger ultimate strain. Observing FB-2 (1.34%) and FB-6 (1.60%), pre-cracking slightly increases the ultimate strain. In addition, it is obvious that all specimens, except the pre-cracking specimen FB-5 and FB-6, have a similar and large initial slope. And with load increase, the slope decreases gradually in different speed, which shows that the strain grows faster and faster and the FFRP carries larger and larger load. It is noteworthy that all specimens, except for FB-5, present obvious elastic recovery (the FRP strain decreased rapidly) after reached the ultimate strain in the load-strain curves and that no debonding was observed in test process in all specimens. Similar tends of load-FRP strain relationship were also found in other strain gauges. This is attributed by the fact that the FFRP ruptured, followed with a rapid decrease in strain.

![Figure 1 (a) Load-deflection responses, and (b) mid-span load-FRP strain relationship](image_url)
Ductility Indices and Energy Absorption Capacities

As for ductility, the ductility index $\mu_{dy}$ is calculated with the mid-span deflection at yield load ($\delta_y$) and the mid-span deflection at ultimate load ($\delta_u$). From Table 1, it is clear that all the FFRP strengthened specimens have higher ductility index than the control specimens and the pre-cracking specimens have the highest ductility index. Thus, the FFRP reinforcement has a significant effect on the improvement of ductility for RC beams. In this study, the energy absorption is calculated using the area under the load-deflection curve to measure the ductility. Among all the specimens, FB-4 has the largest energy absorption (2844.2 kN-mm), while the control specimen CB-1 has the smallest energy absorption (369.7 kN-mm). Obviously, the energy absorption of FB-1 (1196.9 kN-mm) and FB-2 (1853.9 kN-mm) is much larger than that of CB-1. The beam FB-3 (2720.7 kN-mm) and FB-4 (2844.2 kN-mm) has slightly small increase compared to that of CB-2 (2055.1 kN-mm) in energy absorption, respectively. In addition, the specimen FB-5 (1094.8 kN-mm) and FB-6 (1821.8 kN-mm) are slightly smaller than FB-1 and FB-2 in energy absorption due to the influence of pre-cracking. Thus, FFRP strengthening is beneficial for the increase of ductility and energy absorption in RC beams.

Prediction of Ultimate Load-Carrying Capacity Based on ACI 440.2R-08

Table 2 listed the experimental ultimate load ($P_{eu}$) and the predicted ultimate load ($P_{up}$) of FFRP reinforced composite beams from ACI 440.2R-08 (ACI 440.2R-08). It is obvious that all the values of $P_{up}$ / $P_{eu}$ are less than 1, which means that the predicted ultimate load are smaller than that of specimen obtained from the experimental results. The difference between the predicted results and the experimental results ranges from 3.0% to 40.0% for all the strengthened beams. Specimen FB-1, FB-2, FB-5 and FB-6 with 4 layers FFRP laminates have slightly smaller differences between the predicted results and the experimental results, while the specimen FB-3 and FB-4 with 6 layers FFRP laminates have larger differences. In general, the ACI 440.2R-08 design guidelines is relatively conservative with a margin for the ultimate load capacity of FFRP reinforced beams.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Steel reinforcement</th>
<th>Fabric layers</th>
<th>$P_{eu}$ (kN)</th>
<th>$P_{up}$ (kN)</th>
<th>$P_{up} / P_{eu}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>FB-1</td>
<td>C 2 8</td>
<td>4</td>
<td>77.7</td>
<td>75.4</td>
<td>0.971</td>
</tr>
<tr>
<td>FB-2</td>
<td>C 2 8</td>
<td>6</td>
<td>95.0</td>
<td>85.0</td>
<td>0.895</td>
</tr>
<tr>
<td>FB-3</td>
<td>C 2 12</td>
<td>4</td>
<td>164.3</td>
<td>117.4</td>
<td>0.715</td>
</tr>
<tr>
<td>FB-4</td>
<td>C 2 12</td>
<td>6</td>
<td>172.3</td>
<td>126.7</td>
<td>0.735</td>
</tr>
<tr>
<td>FB-5</td>
<td>C 2 8</td>
<td>4</td>
<td>79.0</td>
<td>75.4</td>
<td>0.955</td>
</tr>
<tr>
<td>FB-6</td>
<td>C 2 8</td>
<td>6</td>
<td>98.3</td>
<td>85.0</td>
<td>0.865</td>
</tr>
</tbody>
</table>

Comparison with RC Beams with Externally Bonded Synthetic FRP Plates

Table 3 gives the physical and tensile properties of some typical natural fibres and synthetic fibres reported in the literature. It shows that the tensile strength and modulus of flax fibre are larger compared with the tensile strength and modulus of other typical natural fibres like sisal, hemp and jute fibres. The tensile strength and modulus of flax fibre are close to those of E-glass fibres, but still much lower than those of carbon fibres. However, the elongation at failure of flax fibre is larger than that of E-glass or carbon fibre. Regarding the cost, flax fibre is much than that of E-glass or carbon fibre. In addition, the energy consumption for production of one ton of fibre is 2750 MJ, which is only 8.7% and 0.8% that of E-glass and carbon fibre, respectively. However, the use of natural flax fibres is more cost-effective and energy-effective compared with that of E-glass or carbon fibre.

For axial compression, the confinement effectiveness and confinement ratio of natural flax FRP tube confined concrete were compared with those of G/CFRP confined concrete from study of Yan and Chouw (Yan and Chouw 2014b). The tensile properties of G/CFRP composites used in these studies varies from 381 to 4400 MPa and 38.1 to 580 GPa. The compressive strength of the unconfined concrete (25.7 MPa) for FRP confined concrete is close to those (26.2 MPa to 29.8 MPa) used for G/CFRP confined concrete (Pessiki et al. 1997; Wang and Cheong, 2001 and Shehata 2002). The confinement effectiveness and confinement ratio of G/CFRP confined concrete varies from 1.11 to 2.96 and from 0.033 to 0.604. The obtained confinement effectiveness and confinement ratio of FFRP confined concrete are 1.46-2.08 and 0.210-0.566, respectively. Therefore, in axial compression, the confinement performance of natural FFRP on concrete is close to that of G/CFRP on concrete, even the tensile properties of FFRP composites are much lower than those of G/CFRP composites.

For flexural behaviour, Rahimi and Hutchinson (Rahimi and Hutchinson 2001) investigated RC beams strengthened with externally bonded GFRP, CFRP and mild steel plates by four-point bending. Compared with
the ultimate load of the control RC beam with internal steel reinforcement of 1.68%, the increase in ultimate load due to 4-layer CFRP, 6-layer CFRP, 12-layer GFRP and 3-mm thick steel plate strengthening is 28.0%, 73.3%, 48.2% and 40.9%, respectively. For FFRP strengthened RC beams in this study, compared with the ultimate load (463 kN) of control RC beam with internal steel reinforcement of 0.223%, the increase in ultimate load of RC beams due to 4-layer and 6-layer FFRP plate strengthening is 67.8% and 105.2%, respectively. Compared with the ultimate load (142.3 kN) of control RC beam with internal steel reinforcement of 0.503%, the increase in ultimate load of RC beams due to 4-layer and 6-layer FFRP plate strengthening is 15.5% and 21.1%, respectively. Compared the enhancement in ultimate load of RC beams due to externally bonded CFRP, CFRP and steel plates given in Ref. (Rahimi and Hutchinson 2001) with that due to externally bonded FFRP plate in this study, it can be seen that the improvement in ultimate load of RC beams with similar dimension due to externally bonded natural FFRP plates is comparable to that due to externally bonded GFRP and CFRP and is close to that due to externally bonded CFRP plates. However, it is true that the tensile strength and modulus of natural FFRP plates are significantly lower compared with those of CFRP, GFRP and steel.

**Table 3 Physical and tensile properties and cost of natural fibres and glass fibres (Yan et al. 2014b; Cristaldi et al. 2010)**

<table>
<thead>
<tr>
<th>Fibre type</th>
<th>Relative density (g/cm³)</th>
<th>Tensile strength (MPa)</th>
<th>Elastic modulus (GPa)</th>
<th>Specific modulus (GPa×cm²/g)</th>
<th>Elongation at failure (%)</th>
<th>Cost (USD/kg)</th>
<th>Energy for production (MJ/t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flax</td>
<td>1.4-1.5</td>
<td>343-2000</td>
<td>27.6-103</td>
<td>45</td>
<td>1.2-4.3</td>
<td>0.70</td>
<td>2750</td>
</tr>
<tr>
<td>Sisal</td>
<td>1.33-1.5</td>
<td>363-700</td>
<td>9.0-38</td>
<td>17</td>
<td>2.0-7.0</td>
<td>0.50</td>
<td>2500</td>
</tr>
<tr>
<td>Hemp</td>
<td>1.4-1.5</td>
<td>270-900</td>
<td>23.5-90</td>
<td>40</td>
<td>1.3-5</td>
<td>1.0</td>
<td>4170</td>
</tr>
<tr>
<td>Jute</td>
<td>1.3-1.49</td>
<td>320-800</td>
<td>30</td>
<td>30</td>
<td>1.1-8</td>
<td>0.4</td>
<td>3600</td>
</tr>
<tr>
<td>E-glass</td>
<td>2.2-2.6</td>
<td>800-2000</td>
<td>36-76</td>
<td>16-30</td>
<td>1.8-2.8</td>
<td>2.5</td>
<td>31700</td>
</tr>
<tr>
<td>Carbon</td>
<td>1.6-1.8</td>
<td>1600-3800</td>
<td>80-320</td>
<td>50-178</td>
<td>1.0-1.6</td>
<td>10</td>
<td>355000</td>
</tr>
</tbody>
</table>

**CONCLUSIONS**

1. FFRP strengthening increases the deflection of the specimens remarkably. The increase in deflection is more propounded for beams with a lower steel reinforcement ratio. In addition, beams with more FFRP layers have larger deflection.

2. Both ductility and energy absorption increase significantly due to FFRP strengthening compared to the control beams. The increase in ductility and energy absorption is larger in low-reinforcement ratio beams than that in high-reinforcement ratio beams.

3. The pre-cracking by applying 80% of yield load of the control beam slightly increases the deflection at failure, ductility of the beam and ultimate FRP strain of the strengthened beams, but has no obvious effect on the failure mode, ultimate load capacity and energy absorption.

4. The experimental results of ultimate load-carrying capacity are in general agreement with the predictions that computed according to ACI 440.2R-08 design guidelines. However, the ACI 440.2R-08 design guidelines are relatively conservative with the margin ranged from 3.0% to 40.0%, especially for high-reinforcement ratio beams. When the number of FFRP layer increases, the predictions become less accurate.

5. The comparison with RC beams having similar dimension with externally bonded GFRP, CFRP and steel plates indicates that the enhancement in ultimate lateral load carrying capacity due to natural FFRP plates is close to comparable to the GFRP, CFRP and steel plate strengthened beams, although the tensile strength and modulus of FFRP composites are significantly lower than those of CFRP, GFRP and steel.

In general, this study reveals that natural flax fabric reinforced polymer composites as an environmentally-friendly external reinforcement material can be used to retrofit and/or strengthen exiting deficiently-designed and/or damaged RC structures, which might be further extended for the existing masonry and wood structures.

**ACKNOWLEDGMENTS**

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**REFERENCES**


EXPERIMENTAL STUDY ON SHEAR BEHAVIOR OF RC BEAMS JACKETED BY JUTE-NFRP

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4 Department of Civil Engineering, Kasetsart University, Thailand.

ABSTRACT

Jute-NFRP is remarkably less expensive than conventional FRP, such as CFRP and ARFP, and is less harmful to the environment. This study aims to observe the shear behaviour of RC beams jacketed by Jute fibre sheets, and to develop a new model of shear strength prediction of such RC beams. In the experimental program, a total of six RC beams were tested under a three-point bending load. One control beam was designed with insufficient shear reinforcement. To investigate enhancement of shear strength, five RC beams were strengthened by jacketing one to five layers of Jute-NFRP in shear direction. From the experimental results, shear strength of Jute-NFRP strengthened beams increased in number of NFRP layers. In addition, by shifting the failure mode from brittle shear failure to more ductile failure, it can be demonstrated that Jute-NFRP jacketing is effective for enhancing the shear strength of RC beams approximately 80.2-204%. The experimental results in terms of shear strength were compared with those from existing predicting equations.

KEYWORDS

FRP jacketing, RC beams, natural fibre, jute fibre, shear strengthening, shear strength prediction.

INTRODUCTION

Several reinforced concrete (RC) structures without properly designed shear reinforcement are susceptible to catastrophic shear collapse under severe loading. In recent years, Fibre Reinforced Polymer (FRP) sheets have become increasingly in use for shear strengthening RC structures owing to their high stiffness, strength-to-weight ratios and design flexibility (Vistasp and Lei 2000). Conventional FRPs such as Carbon FRP (CFRP) and Aramid FRP (AFRP) are effective in enhancing shear strength, but their applications in structures is limited mainly by their high initial cost and relatively high environmental impact (Mohanty et al. 2000). Instead of those synthetic FRPs, use of Natural Fibre Reinforced Polymer (NFRP) such as Jute FRP has gained popularity in engineering applications since they are more cost effective and environmental friendly while maintaining high mechanical properties comparable to those of CFRP and Glass FRP (GFRP) used as reinforcement (Sen and Reddy 2013). Although Jute NFRP can be applied in concrete structures, its structural performance for strengthening RC structures has not been clearly investigated to ensure its structural application and safety.

Jute-NFRP sheets usually contain fibres in two orthogonal directions, termed as warp and weft directions. It was found in this study that tensile strength, elastic modulus and fracturing strain are different in the different fibre direction (Tidarut 2015). In the weft direction, their tensile strength and stiffness tend to be stronger than that in the warp direction. This research is therefore aimed to study shear strengthening of RC beams jacketed by Jute-NFRP in only WEFT direction owing to its high strength and stiffness. To examine the structural performances of RC beams jacketed by Jute-NFRP, a total six rectangular beams were prepared including one control specimen without Jute-NFRP jacketing. Five specimens were jacketed with U-wrap shape in which the weft direction was set normal to the member axis with one to five number of Jute-NFRP’s layers. From the experimental results, beams with jute-NFRP jacketing all failed in shear, with improving shear strength. This proved that jute-NFRP effectively increase shear strength with increasing number of layers. This fact proved that Jute-NFRP can
effectively increase shear strength. In addition, the experimental shear strength of RC beams was compared with that obtained from design equation of JSCE code in order to investigate its applicability.

EXPERIMENTAL PROGRAMS

Material Properties

Concrete and Steel Reinforcement

Ready-mixed concrete with the compressive strength of 34 MPa at 14 days was used in the experiment. Steel moulds were prepared, and the size was adjusted with wooden plates placed perpendicularly to the mould’s bottom. Longitudinal reinforcement with diameter of 25 mm had yielding strength of 400 MPa, whereas stirrup with diameter of 6 mm had yielding strength of 240 MPa. Table 1 summarizes the properties of steel reinforcement.

Table 1 Properties of steel reinforcement

<table>
<thead>
<tr>
<th>Type</th>
<th>Diameter (mm)</th>
<th>Area (mm²)</th>
<th>$f_y$ (MPa)</th>
<th>$E_s$ (MPa)</th>
<th>$\varepsilon_y$ (με)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal Reinforcement</td>
<td>25</td>
<td>491.07</td>
<td>400</td>
<td>183,000</td>
<td>2200</td>
</tr>
<tr>
<td>Stirrup</td>
<td>6</td>
<td>31.67</td>
<td>240</td>
<td>191,000</td>
<td>1300</td>
</tr>
</tbody>
</table>

$* f_y =$ yielding strength, $E_s =$ elastic modulus and $\varepsilon_y =$ yielding strain.

Jute-NFRP and Epoxy Resin

Tensile properties of Jute fibres are imperative in order to consider them as reinforcement. According to Horsangchai et al. 2016, coupon test of Jute-NFRP was conducted considering fibre alignment in both warp and weft directions with fibre density of 1.41 g/cm³. Coupon specimens for WARP and WEFT were denoted as WARP1 to WARP5 and WEFT1 to WEFT5, respectively. From the coupon test, tensile strength, elastic modulus and rupture strain were obtained for ten coupon specimens are shown in Table 2 and Figure 1. The properties of the epoxy resin used in the test to uniformly gather the fibres are shown in Table 3.

Table 2 Properties of the Jute-NFRP

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Width (mm)</th>
<th>Length (mm)</th>
<th>Design thickness (mm)</th>
<th>Tensile Strength (MPa)</th>
<th>Initial Elastic Modulus $E_1$ (MPa)</th>
<th>2nd stage Elastic Modulus $E_2$ (MPa)</th>
<th>Ultimate Strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WARP1</td>
<td>31.5</td>
<td>79.2</td>
<td>0.18</td>
<td>168</td>
<td>11,711</td>
<td>5,413</td>
<td>1.9</td>
</tr>
<tr>
<td>WARP2</td>
<td>30.5</td>
<td>79.2</td>
<td>0.18</td>
<td>167</td>
<td>8,465</td>
<td>5,618</td>
<td>2.6</td>
</tr>
<tr>
<td>WARP3</td>
<td>30.3</td>
<td>79.0</td>
<td>0.18</td>
<td>138</td>
<td>9,345</td>
<td>4,885</td>
<td>1.9</td>
</tr>
<tr>
<td>WARP4</td>
<td>32.7</td>
<td>79.0</td>
<td>0.18</td>
<td>132</td>
<td>8,819</td>
<td>3,650</td>
<td>1.9</td>
</tr>
<tr>
<td>WARP5</td>
<td>33.8</td>
<td>79.1</td>
<td>0.18</td>
<td>172</td>
<td>8,706</td>
<td>5,324</td>
<td>2.5</td>
</tr>
<tr>
<td>WARP- Average</td>
<td>31.8</td>
<td>79.1</td>
<td>0.18</td>
<td>155</td>
<td>9,409</td>
<td>4,978</td>
<td>2.2</td>
</tr>
<tr>
<td>WEFT1</td>
<td>31.4</td>
<td>80.0</td>
<td>0.18</td>
<td>229</td>
<td>15,666</td>
<td>10,364</td>
<td>1.9</td>
</tr>
<tr>
<td>WEFT2</td>
<td>31.7</td>
<td>78.9</td>
<td>0.18</td>
<td>134</td>
<td>19,361</td>
<td>14,250</td>
<td>0.9</td>
</tr>
<tr>
<td>WEFT3</td>
<td>29.8</td>
<td>78.4</td>
<td>0.18</td>
<td>146</td>
<td>16,033</td>
<td>11,263</td>
<td>0.8</td>
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<tr>
<td>WEFT4</td>
<td>31.0</td>
<td>78.6</td>
<td>0.18</td>
<td>203</td>
<td>15,282</td>
<td>13,409</td>
<td>1.6</td>
</tr>
<tr>
<td>WEFT5</td>
<td>30.2</td>
<td>78.5</td>
<td>0.18</td>
<td>160</td>
<td>15,407</td>
<td>9,885</td>
<td>1.2</td>
</tr>
<tr>
<td>WEFT- Average</td>
<td>30.8</td>
<td>78.9</td>
<td>0.18</td>
<td>174</td>
<td>16,350</td>
<td>11,834</td>
<td>1.3</td>
</tr>
</tbody>
</table>

Figure 1 Tensile tests of coupon specimens (Horsangchai et al. 2016)
Table 3 Properties of the epoxy resin

<table>
<thead>
<tr>
<th>Name</th>
<th>No.</th>
<th>Compressive strength (MPa)</th>
<th>Modulus of elasticity (MPa)</th>
<th>Shear strength (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>Consumption (kg/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S&amp;P Resin</td>
<td>220</td>
<td>&gt;70</td>
<td>&gt;7,100</td>
<td>&gt;26</td>
<td>&gt;14</td>
<td>1.75</td>
</tr>
</tbody>
</table>

RC Beam Test

All six beams were tested under static loading until their failure. Each beam had a shear span \( (a) \) of 500 mm, and cross section of 150 mm × 300 mm \((b \times h)\) with chamfering corner of 13 mm in radius to prevent rupture of fibre sheet and stress concentration at the corners. The shear-span to effective ratio \((a/d)\) was 2.0, representing normal RC beams. In Figure 2, two longitudinal reinforcements were placed parallel to each other in the compression zone, whereas two layers of two longitudinal reinforcements were placed in the tension zone. The stirrups were placed along the main steel reinforcements with spacing of 150 mm. In all six beams, the longitudinal reinforcement and stirrup ratios were 5.14% and 0.28% respectively, and volumetric ratio of Jute-NFRP sheet varied from 0.24% to 1.19% with increasing number of Jute-NFRP layers. The parameters were the number of jute-NFRP layers. Table 2 shows the details of six beams. Strain gauges were attached on stirrups, longitudinal reinforcement and jute-NFRP sheets to measure their strains, especially along where shear cracks were expected to occur. Linear Variable Differential Transducer (LVDT) was used to measure vertical deformation at mid-span. Figure 3 shows the preparation of RC beams and Table 4 shows the details of tested RC beam. The RC beams were denoted as Control for unstrengthened one and WEFT1-WEFT5 for strengthened ones with 1 to 5 layers of Jute-NFRP.

![Figure 2 Details of beam and its cross section](image)

![Figure 3 Preparation of RC beams](image)

Table 4 Details of RC beam specimens

<table>
<thead>
<tr>
<th>Name</th>
<th>No. of layers</th>
<th>( b ) (mm)</th>
<th>( d ) (mm)</th>
<th>( h ) (mm)</th>
<th>( a ) (mm)</th>
<th>( a/d ) (−)</th>
<th>( \rho_{st} ) (%)</th>
<th>( \rho_w ) (%)</th>
<th>( \rho_f ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>0</td>
<td>150</td>
<td>255</td>
<td>300</td>
<td>500</td>
<td>2.0</td>
<td>5.14</td>
<td>0.28</td>
<td>0.00</td>
</tr>
<tr>
<td>WEFT1</td>
<td>1</td>
<td>150</td>
<td>255</td>
<td>300</td>
<td>500</td>
<td>2.0</td>
<td>5.14</td>
<td>0.28</td>
<td>0.24</td>
</tr>
<tr>
<td>WEFT2</td>
<td>2</td>
<td>150</td>
<td>255</td>
<td>300</td>
<td>500</td>
<td>2.0</td>
<td>5.14</td>
<td>0.28</td>
<td>0.47</td>
</tr>
<tr>
<td>WEFT3</td>
<td>3</td>
<td>150</td>
<td>255</td>
<td>300</td>
<td>500</td>
<td>2.0</td>
<td>5.14</td>
<td>0.28</td>
<td>0.71</td>
</tr>
<tr>
<td>WEFT4</td>
<td>4</td>
<td>150</td>
<td>255</td>
<td>300</td>
<td>500</td>
<td>2.0</td>
<td>5.14</td>
<td>0.28</td>
<td>0.95</td>
</tr>
<tr>
<td>WEFT5</td>
<td>5</td>
<td>150</td>
<td>255</td>
<td>300</td>
<td>500</td>
<td>2.0</td>
<td>5.14</td>
<td>0.28</td>
<td>1.19</td>
</tr>
</tbody>
</table>

* Note that \( a/d \) = shear-span to effective ratio, \( \rho_{st} \) = longitudinal reinforcement ratio, \( \rho_w \) = stirrup ratio, \( \rho_f \) = Jute-NFRP sheet’s ratio
EXPERIMENTAL RESULTS

Failure Modes
Failure modes before and after removing Jute-NFRP sheets are shown in Figure 4. Figures on the left side show failure modes before removal of Jute-NFRP, and those on the right show failure modes after removal of Jute-NFRP. The control beam failed in anchorage failure with some minor shear cracks. It may be because the main longitudinal reinforcement did not have enough anchorage or hooked length and such reinforcement significantly moved during the loading test. For WEFT 1 and WEFT2, they failed in shear with rupture of Jute-NFRP along the shear cracks. It can also be seen that Jute-NFRP strengthened beams did not fail in anchorage failure since the Jute-NFRP attachment could prevent the movement of the longitudinal reinforcements inside. For WEFT3, WEFT4 and WEFT5, they failed in shear compression failure while no major cracks were observed on the beams’ surfaces and Jute-NFRP sheets. However, some minor cracks were seen on the beams.

![Figure 4 Failure modes of RC beams](image)

Shear Force Components
Relationships between total shear force and vertical deformation at mid-span of RC beams are shown in Figure 5a. Shear strengths of Jute-NFRP jacketed beams are enhanced when increasing the number of Jute-NFRP layers. Deformations at the peak load also increase gradually when increasing the number of layers of Jute-NFRP. Maximum total shear strength and corresponding deformation of strengthened beams are higher than those of control beam, as shown in Fig. 5a. The enhanced total shear strength increases with the amount of Jute-NFRP, by varying from 80.2% to 204% for WEFT1 to WEFT5. This total shear force ($V_{tot}$) consists of shear forces carried by concrete ($V_c$), stirrup ($V_s$) and NFRP sheet ($V_f$), as shown in Eq. (1). Shear forces carried by stirrup and FRP sheet are calculated using strains of stirrup and FRP sheet measured along shear crack, as shown in Figure 2 (see...
strain gauges in circles). Shear force carried by concrete \( (V_c) \) can be obtained by subtracting the shear force carried by stirrup and FRP from the total shear force with \( (V_{tot} - V_s - V_f) \). Figure 5b shows shear forces carried by stirrups and deformations.

Figure 5b shows the relationship between shear force carried by stirrup \( (V_s) \) and deformation at mid-span. Beams with more NFRP layers show smaller \( V_s \)-value at the same deformation. This is because the NFRP delayed the elongation and yielding of stirrups. On the other hand, \( V_s \)-behavior did not fully correspond to number of layers, namely different behavior between 1-2 layers and 3-5 layers. This is owing to the difference of failure mode. WEFT1 and WEFT2 exhibited shear failure, whereas WEFT3, WEFT4 and WEFT5 failed in shear compression failure. This shear compression failure does not show major cracks as shear failure does. In other words, crack opening tends to be smaller in shear compression failure than that of shear failure, resulting in the difference of stirrup elongation. Stirrups in beams with larger number of layers showed larger deformation. It can be observed from \( V_s \)-behavior also that more number of NFRP layers give larger ductility to RC beams.

**Shear forces carried by Jute-NFRP and Concrete**

Figure 6a shows the relationship between shear strength carried by Jute-NFRP \( (V_f) \) and deformation at mid-span. Steady increase in \( V_f \)-value with layer increase can be observed from layers 1 to 4. However, WEFT5 shows small \( V_f \)-value because the strain measured from strain gauges on the fiber sheet could not record the local large strain, so called localization of strain. Other than that, it can be concluded that more layers of NFRP could effectively contribute to shear forces carried by Jute-NFRP and ductility as peak shifts in larger deformation.

Figure 6b shows the relationship between shear strength carried by concrete \( (V_c) \) and deformation at mid-span. \( V_c \)-value tends to be larger when increasing number of NFRP layers. This is because NFRP confined the beam, and more layers of NFRP contributed to larger confinement effect. Observed crack opening is also delayed owing to NFRP. \( V_c \)-value constantly increased with more layers of NFRP, except WEFT4 specimen. It may be due to debonding failure occurring at the compression failure zone. Such debonding could propagate depending on bonding strength of NFRP, which varies in each specimen. Furthermore, \( V_c \)-value is calculated by subtraction of \( V_s \) and \( V_f \) from \( V_{tot} \). It can be explained that as \( V_f \) increased in WEFT3 to WEFT4 while \( V_{tot} \) did not increase, leading to decrease in \( V_c \)-value.

**DISCUSSIONS ON APPLICABILITY OF EXISTING DESIGN EQUATIONS**

To predict shear strength of beam strengthened by Jute-NFRP, shear strength obtained from experiment and JSCE code (2002) are compared. Total shear strength can be calculated from Eq. (1). The concrete and stirrup contributions to shear strength can be calculated as follows:

\[
V_c = 0.2\sqrt{f'_c/1000/d\sqrt{100}}\rho_w (bd) \quad (1)
\]

\[
V_s = A_w f_{we} (\sin \alpha + \cos \alpha) z / s \quad (2)
\]

where \( f'_c \) is the compressive strength of concrete; \( b \) is the width of member; \( d \) is the effective depth of member; \( \rho_w \) is the ratio of transverse steel reinforcement; \( A_w \) is the cross-sectional area of transverse steel reinforcement; \( f_{we} \) is the yielding strength of transverse reinforcement; \( \alpha \) is the angle of transverse steel reinforcement to the member’s axis; and \( z \) is \( d/1.15 \); \( s \) is the spacing of shear reinforcement. The shear contribution provided by FRP sheet is its tensile capacity and is computed based on the shear reinforcing efficiency of the FRP sheet \( (K) \) as shown in Eq. (3).
\[ V_f = KA_f f_{\mu} (\sin \alpha_f + \cos \alpha_f) \frac{\rho_v}{s_f} \]

where \( K = 1.68 - 0.67R \) in which \( 0.4 \leq K \leq 0.8 \) and \( R = \left( \frac{f_{\mu}}{E_f} \right)^{\frac{1}{14}} \left( \frac{f_{\mu}}{E_f} \right)^{\frac{2}{13}} \) in which \( 0.5 \leq R \leq 2.0 \); \( A_f \) is the cross-sectional area of the FRP sheet; \( f_{\mu} \) is the design tensile strength of the FRP sheet (N/mm²); \( s_f \) is the spacing of the FRP sheet; \( E_f \) is the modulus of elasticity of the FRP sheet (kN/mm²); \( \rho_v \) is the volumetric ratio of the FRP sheet; and \( \alpha_f \) is the angle of the FRP sheet to the member axis.

Comparison of shear strength between JSCE code and experiment is shown in Table 5 and Figure 7. \( V_{\text{tot}} \) of JSCE code is quite conservative (range of error = -23.2% to +6.0%). It can be seen that each shear component (e.g., \( V_c \), \( V_s \), and \( V_f \)) also show discrepancies between experimental and predicted values. For shear forces carried by concrete \( (V_c) \), the predicted values are underestimated when compared to the experimental ones. This is partly because the \( V_c \)-equation of JSCE does not consider the confinement effect by jacketing sheet. On the contrary, predicted shear forces carried by Jute-NFRP \( (V_f) \) show overestimated values since the K-factor in JSCE code is based on behavior of conventional FRP such as CFRP and AFRP. In conclusion, the predicting equation for \( V_c \) and \( V_f \) should be modified in case of NFRP for higher accuracy in the future research work.

### Table 5 Shear force component at peak obtained from experiment and JSCE code (2002)

<table>
<thead>
<tr>
<th>Description</th>
<th>Experiment (at Peak)</th>
<th>JSCE (at Peak)</th>
<th>% error of ( V_{\text{tot}} ) between Experiment-Code</th>
<th>% increase of ( V_{\text{tot}} ) between Control and WEFT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>68.1 60.7 7.4 0.0</td>
<td>72.2 49.7 22.5 0.0</td>
<td>+6.0</td>
<td>-</td>
</tr>
<tr>
<td>WEFT1</td>
<td>122.7 63.1 53.2 6.4</td>
<td>94.2 49.7 22.5 22.0</td>
<td>-23.2</td>
<td>80.2</td>
</tr>
<tr>
<td>WEFT2</td>
<td>142.7 82.5 50.0 10.3</td>
<td>116.2 49.7 22.5 44.0</td>
<td>-18.6</td>
<td>109.5</td>
</tr>
<tr>
<td>WEFT3</td>
<td>156.7 104.5 36.5 15.7</td>
<td>138.2 49.7 22.5 66.0</td>
<td>-11.8</td>
<td>130.1</td>
</tr>
<tr>
<td>WEFT4</td>
<td>153.2 77.4 39.2 36.5</td>
<td>160.2 49.7 22.5 88.0</td>
<td>+4.6</td>
<td>125.0</td>
</tr>
<tr>
<td>WEFT5</td>
<td>207.2 147.2 34.8 25.2</td>
<td>182.1 49.7 22.5 109.9</td>
<td>-12.1</td>
<td>204.3</td>
</tr>
</tbody>
</table>

Figure 7 Comparison between shear strength from experiment and JSCE code (2002)

**CONCLUSIONS**

1. Jute-NFRP is an ideal and effective material to strengthen RC beams with low cost. It enhances shear strength of RC beams. However, more study should be performed in terms of the in-depth behavior of each shear force components.

2. Increasing the number of Jute-NFRP sheet layer can increase the total shear strength and concrete shear strength. The enhancements of total shear strength are from 80.2% to 204% for WEFT1 to WEFT5 beams.

3. The design equation in JSCE code slightly underestimates the shear strength of Jute-NFRP strengthened beams with range of error between -23.2% and +6.0%. However, each shear component, especially shear forces carried by concrete \( V_c \) and shear forces carried by NFRP \( V_f \) should also be revised taking into account confinement effect and various types of NFRP.

**ACKNOWLEDGMENTS**

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REFERENCES


SHEAR BEHAVIOR OF RC BEAMS REINFORCED BY NATURAL COMPOSITES MATERIALS (FLAX TECHNIC FIBER): EXPERIMENTAL-MODELING STUDY

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ABSTRACT

This paper presents the results of an experimental program on RC beams reinforced by natural composite (Flax technic fiber) for shear resistance by External Bonding method (EB) and modeling results by finite element method in using Abaqus software. A total of 11 beams divided in two groups with the same geometry and with different concrete strength was tested. The composite’s strain was monitored by using strain gauges bonded on high stress zone associated with digital image correlation (DIC) method. The beams have a rectangular section and a 2m span. The following parameters were investigated: (i) ratio of reinforcement by FRP (continuous and non-continuous), (ii) interaction between FRP and transverse steel (stirrup). The results show that the shear resistance capacity of beams reinforced by Flax technic composite significantly increases, from 11% to 28%. The results also show that the presence of stirrups decreases the contribution of composite to the shear resistance of beams. On the modeling using finite element method, the results showed a good similarity / convergence with experimental results of control beams and also of reinforced beams with the perfect bonding interface of composite-concrete. The results of the correlation image method also present good agreement with modeling results obtained by Abaqus.

KEYWORDS
FRP, RC beams, shear strengthening, Flax technic, natural composite.

INTRODUCTION

In the construction, the composite materials are very popular for the reinforcement of structural or for repairing old structures of external bonding method. The current composite materials which were used for reinforcement of structure are: Carbon-fiber-reinforced polymer (CFRP), Glass-fiber-reinforced polymer (GFRP)… This type of composite presents several advantages like: corrosion resistance, high resistance, easy for application on structure… However, these materials are of petrol origin, thus the production has an unfavorable impact on the environment, and these materials are not recyclable and request millions years for degradation. Today, to answer the question of sustainable development requirement, it is necessary to find a new eco-friendly material that can replace these materials. In collaboration with the project FABILIN at laboratory of composites materials for construction (LMC²), the high-performance flax fiber was studied in this paper. This material was already used in several domains: automobile industries, airline industries to replace some elements of automobiles, planes…

To check the efficiency of Flax fiber on reinforcement of RC structures, the program of test is done on shear strengthening RC beams reinforced by this fiber by external bonding method. The principal objective of this study is the evaluation of the capacity of Flax fiber in shear reinforcement of RC beams. These investigated parameters are: (i) ratio of reinforcement of FRP (continuous vs non-continuous), (ii) strength of concrete, (iii) interaction of FRP and transverse steel (stirrup).

RESEARCH SIGNIFICANCE

The shear reinforcement of RC beams by external CFRP or GFRP bonding method was investigated by several authors (Teng et al., 2009), (Khalifa et al., 1998), (Khalifa and Nanni, 2000),… The failure mode obtained in these experiments was often by debonding of composite reinforcement, so it limit the full used of the high-capacity of CFRP and GFRP. By using natural composites materials for reinforcement RC structure, some studies investigated the flexure reinforcement of RC beam (Bharath and Reddy, 2015),(Sen and Reddy, 2013),(Sandeep kumar et al.,
The results showed a significant enhancement of flexure capacity RC beam. This study will conduct on the shear reinforcement of RC beams by using flax technic fiber.

**EXPERIMENTAL PROGRAM**

**Description of specimens**

A total of 11 samples beams which was divided in two groups (group A and group B) were tested in flexion of three points. All beams have the same geometric configuration: 2m of span, rectangular section 150x250mm², the loading point was found from 0.5m of support for high shear force in this part which considered as zone of the test (Figure 1), the force part of 1.5m length was reinforced with stirrups Ø6 of 50mm spacing to ensure that there is no shear failure in this part. to ensure that there is no flexure failure of control beams during the test, three bars Ø20 was used for longitudinal reinforcement. The differences for each beams were the spacing of stirrups in the zone of test, no stirrups and stirrups of 150mm spacing for group A (A0 and A150), and stirrups at 180mm for all beams of group B (B180); and the strength of concrete for group A and group B were 40MPa and 30MPa respectively.

![Figure 1](image-url) a) Geometry of test, b) Cross section with stirrups, c) Cross section with no stirrups, d) Cross section with FRP

One beam of each series is tested as control beam, and the remaining beams were reinforced by flax technic fiber in using external bonding method –U jacket, and tested as reinforced beams. The configuration of each beam are showed in Table 1:

<table>
<thead>
<tr>
<th>Table 1 Configuration of all beams</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image-url" alt="Diagram of beams" /></td>
</tr>
</tbody>
</table>

Note: for reinforced beams, example beam A150-200-3LFRP means: A150-group of beam and spacing of stirrups in zone of test, 200: spacing of sheet composite (‘’cont’’ mean reinforcement continuous), 3LFRP: number of layers LFRP.

**Materials**

Concrete used is prepared in laboratory, standard compression tests on control cylinders revealed a 28-day concrete compressive strength of 40MPa on average for group A and 30MPa on average for group B. The steel reinforcing bars (Ø20, Ø6, and Ø10) is tested in tension according to ASTM A 370 with modulus of elasticity of
200 GPa and yield stress of 520 MPa on average. The mechanical properties of flax technic fiber are provided by tensile test and are showed in Table 2. There are three types of flax’s fibers used for reinforcement: sheet bidirectional 90° (S600 is used for beams A0-cont-2LFRP, A150-cont-2LFRP) and sheet bidirectional 45° (BD700 is used for beam B180-cont-2LFRP-45°) for continuous reinforcement, and strip of 100mm large of tissue unidirectional (UD220) for reinforcement non-continuous. The mechanical properties of composite are tested by tensile test according ISO 527-1. The obtained results are showed in the Table 3.

<table>
<thead>
<tr>
<th>Table 2 Mechanicals properties of flax fiber</th>
<th>Table 3 Mechanicals properties of composites</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mechanicals properties ISO 527 VARIM process &amp; epoxy resin-Vf=40%</td>
<td>Composite</td>
</tr>
<tr>
<td>Maximum tensile stress</td>
<td>550MPa</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>32GPa</td>
</tr>
<tr>
<td>Elongation at tensile failure</td>
<td>1.7%</td>
</tr>
</tbody>
</table>

Test setup and instrumentation

All beams were tested by three points bending test in using hydraulic jack which having a capacity of 500kN. The test was conducted under displacement controlled rate of 1mm/min. The displacements of mid-span and loading point were captured in using LVDT sensors. All data is acquired and recorded in acquisition system VIHAY 5000.

Results and discussion

In Table 4, the contributions in shear resistance of beam from concrete, transverse steel reinforcement, and LFRP reinforcement at ultimate load were showed. Note that the values in this table were obtained based on following assumptions of Eurocode 2: a) the shear resistance of concrete is constant regardless the presence of stirrups or LFRP, b) the shear resistance of stirrups is constant whether the beam is reinforced by LFRP or not. For group A, the results show that the presence of LFRP reinforcement enhance significantly the shear resistance of beam. The maximum gain of the shear contribution of LFRP is 24% on beam A0-cont-2LFRP without stirrups reinforcement and with only reinforced continuous by LFRP. For beams with stirrups reinforcement, the shear gain is 15% for beam A150-cont-2LFRP (with LFRP continuous), and 11% for both A150-150-3LFRP and A150-200-3LFRP (with strip of LFRP). The presence of stirrups decreased the contribution of LFRP for shear resistance of beam. This observation was confirmed in researches of (Mofidi, 2012), (Bousselham and Chaallal, 2006),…For group B, the contribution of LFRP for shear resistance of beams is again confirmed: 28% on beam B180-cont-2LFRP-45° with LFRP reinforcement continuous, and 10%, 19%, 15% on beams B180-200-2LFRP, B180-200-4LFRP, B180-150-4LFRP respectively. For the failure mode, all beams were failed by shear excluding two beams A150-cont-2LFRP and A150-150-3LFRP which failed by flexure.

Figure 2 Load - displacement (group A)  
Figure 3 Load - displacement (group B)
Figure 2 and Figure 3 present the relation between load and displacement for group A and B respectively. At the first elastic phase, all beams have similar values of rigidity. For group A, the shear crack starts at the load about of 75kN with an angle 40° to the beam’s axis. Beams A0-Ref and A0-cont-2LFRP without stirrups reinforcement shows the brittle behavior; as soon as the load reaches the ultimate load, the failure occurs. However, in series A150, with the addition reinforcement of stirrups, the post-failure behavior is more ductile. So with the stirrups reinforcement, post-failure behavior of the beam changes to more ductile mode. This result validates what was observed by (Mofidi, 2012). For group B with stirrups reinforcement of 180mm spacing, all results show a similar elastic behavior up to loading of 55kN. When shear cracks start, the rigidity of beams changes. In this test, the load then continues to increase up to failure.

All beams, reinforced by LFRP, which are failed by shear, present the deboning of FRP (pulling some minor part of concrete away with). No rupture of LFRP is observed.

### Table 4 Experimental results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>n:</th>
<th>D (g/m²)</th>
<th>L_{max}(kN)</th>
<th>V_s(kN)</th>
<th>V_c(kN)</th>
<th>V_t(kN)</th>
<th>\frac{V_f}{V_{Ref}} %</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>A0-Ref</td>
<td>-</td>
<td>-</td>
<td>159</td>
<td>119</td>
<td>119</td>
<td>-</td>
<td>-</td>
<td>shear</td>
</tr>
<tr>
<td>A150-Ref</td>
<td>-</td>
<td>-</td>
<td>173</td>
<td>129</td>
<td>119</td>
<td>10</td>
<td>-</td>
<td>shear</td>
</tr>
<tr>
<td>A0-cont-2LFRP</td>
<td>2</td>
<td>600</td>
<td>210</td>
<td>157</td>
<td>119</td>
<td>-</td>
<td>38</td>
<td>24% shear</td>
</tr>
<tr>
<td>A150-cont-2LFRP</td>
<td>2</td>
<td>600</td>
<td>203</td>
<td>152</td>
<td>119</td>
<td>10</td>
<td>23</td>
<td>15% flexure</td>
</tr>
<tr>
<td>A150-200-3LFRP</td>
<td>3</td>
<td>220</td>
<td>194</td>
<td>145</td>
<td>119</td>
<td>10</td>
<td>16</td>
<td>11% shear</td>
</tr>
<tr>
<td>A150-150-3LFRP</td>
<td>3</td>
<td>320</td>
<td>194</td>
<td>145</td>
<td>119</td>
<td>16</td>
<td>16</td>
<td>11% flexure</td>
</tr>
<tr>
<td>B180-Ref</td>
<td>-</td>
<td>-</td>
<td>89</td>
<td>67</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>shear</td>
</tr>
<tr>
<td>B180-cont-2LFRP</td>
<td>2</td>
<td>700</td>
<td>115</td>
<td>86</td>
<td>67</td>
<td>-</td>
<td>19</td>
<td>28% shear</td>
</tr>
<tr>
<td>B180-200-2LFRP</td>
<td>2</td>
<td>220</td>
<td>99</td>
<td>74</td>
<td>67</td>
<td>-</td>
<td>7</td>
<td>10% shear</td>
</tr>
<tr>
<td>B180-200-4LFRP</td>
<td>4</td>
<td>220</td>
<td>107</td>
<td>80</td>
<td>67</td>
<td>-</td>
<td>10</td>
<td>15% shear</td>
</tr>
<tr>
<td>B180-150-4LFRP</td>
<td>4</td>
<td>220</td>
<td>103</td>
<td>77</td>
<td>67</td>
<td>-</td>
<td>13</td>
<td>19% shear</td>
</tr>
</tbody>
</table>

n: Number of LFRP layer
D: Density of LFRP
L_{max}: Load at failure
V_s: Shear resistance due concrete
V_t: Total shear resistance
V_c: Shear resistance due steel
V_{Ref}: Shear resistance due LFRP

### MODELING

#### Geometric and materials.

In this part, only modeling results of four beams of group A: control beams and reinforced beams by LFRP continuous are presented in using finite elements method using Abaqus software. Two under-software: Abaqus/Standard (implicit) and Abaqus/explicit are used for modeling of quasi-static problem for comparison of results.

The 3D model is used for three-point-bending test of RC beams under bi-axial loading. The boundary condition and force applications are predefined and the symmetries of geometry and load are used to simulate the model structure and thus save computing times. For concrete, elements type C3D4 with linear function is chosen from Abaqus element library. T3D2 truss-elements-lying embedded in the concrete volume-model stirrups and longitudinal bars. Concrete represent the “host element” in the applied concept of “embedded elements”. Thus no bond slip between reinforced and concrete is assumed. FRP is modeled as elements type S3R with using the perfect bonding with concrete surface. For the boundary condition, verticals direction constraints were blocked at two lines where the supports are on. Loading is applied by displacement controlled with function “smooth step” on surface of loading point to avoid dynamic effect on Abaqus/Explicit. The step time in Abaqus/Explicit is chosen 0.1s to assure the quasi-static problem.

The complex, nonlinear behavior of concrete is described by elastic-yielding damage model ‘concrete damage plasticity’. This model requires the following material functions: stress-strain relations for the uni-axial behaviors under compressive as well as tensile loading including cyclic un-and reloading, and functions for the evolution of the damage variables under compressive and tensile loading. The compressive strength is as in the experimental work: 40MPa. $E_c$ and $f_{ct}$ were calculated by Eurocode:

$$E_c = 22 \left( \frac{f_{cm}}{10} \right)^{0.3}$$  \hspace{1cm}  (1)$$f_{ct} = 0.30 f_c^{2/3}$$  \hspace{1cm}  (2)

The stress–strain relationship proposed by (Wang and Hsu, 2001) is used to model the uni-axial compressive stress–strain curve (Eq.3, Eq.4) and tensile stress–strain curve (Eq.5, Eq.6) for concrete:

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\[
\sigma_c = f_{cm} \left[ 2 \left( \frac{\epsilon_c}{\epsilon_{c1}} \right) - \left( \frac{\epsilon_c}{\epsilon_{c1}} \right)^2 \right] \text{ if } \frac{\epsilon_c}{\epsilon_{c1}} \leq 1
\]

(3) \[ \sigma_c = f_{cm} \left[ 1 - \left( \frac{\epsilon_c}{\epsilon_{c1}} - 1 \right)^2 \right] \text{ if } \frac{\epsilon_c}{\epsilon_{c1}} \leq 1 \]

(4)

\[ \sigma = E \epsilon, \text{ si } \epsilon \leq \epsilon_{\text{rupture}} \]

(5) \[ \sigma = f_{ct} \left( \frac{\epsilon_{\text{rupture}}}{E} \right), \text{ si } \epsilon \geq \epsilon_{\text{rupture}} \]

(6)

The behavior of steel reinforcement is assumed perfect elasto-yielding and behavior of LFRP is considered elastic behavior up to failure.

**Results and discussion**

The load-displacement curves obtained for the control beams and the retrofitted beams from experimental and FEM analysis are shown in Figure 4, Figure 5, Figure 6, and Figure 7. Two different methods of calculus is retained implicit and explicit. There is good agreement between explicit method and implicit method results and also between FEM and experimental results for the control beams. For the retrofitted beams by LFRP, the FEM analysis shows a good coherence with experimental results.

![Figure 8 Failure mode: a) A0-Ref, b) A150-Ref](image)
Figure 8 shows the failure mode of control beams: A0-Ref and A150-Ref for the numerical and experimental results. In general, the cracks obtained in the experiments and in the modeling are similar, which indicates that the model can capture the mechanisms of failure in the beams. For the retrofitted beams, the cracks cannot capture by photo because of composite glued on surface of beams. However, the composite’s strain can be measured by the correlation image method. The photos were captured during the test an analyzed with software Icasoft which based on analyses of pixel matrix. Figure 9 shows the result of correction of image and numerical result for the strain of composite. The spec of strain on composite obtained by numerical analysis and of correlation image method are similar. The maximum strain on composite of numerical analysis and correction of image method are 1.1% and 1.0% respectively.

![Image](image.png)

Figure 9 : Failure mode beam A0-cont-2LFRP: a) correction of image result, b) numerical result

**CONCLUSION**

In this study, Flax technic fiber presents a significant effect in the shear reinforcement of RC beam. The modeling by FEM-Abaqus shows good agreement with experimental results. The following conclusions can be drawn from this study:

- The shear reinforcement by Flax technic composite enhances a significant of capacity of shear resistance of RC beams from 11% to 28%.
- The reinforcement of stirrups decreases the contribution of composite in shear resistance of beams, and changes the post-failure behavior of beams.
- The FEM model can capture the mechanisms of fracture in the control beams and also in the reinforced beam by FRP with perfect bonding interface.

**REFERENCES**


BEHAVIOR OF CONCRETE CONFINED WITH NATURAL FIBRE REINFORCED POLYMER

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ABSTRACT

This study aims to establish an in-depth understanding of the compressive behaviour of concrete confined with NFRP. The coupon test of the Jute, Hemp and Cotton NFRP was conducted to obtain its elastic modulus and tensile strength. A total of 63 concrete cylinders were jacketed by the Jute NFRP with a varying number of layers (1 to 6 layers) and the fibre alignment in only the WEFT direction. From the experimental results, the compressive strength of the confined concrete in WEFT direction significantly increases to up to 28%, 42%, and 25% for Cotton, Jute and Hemp confinement, respectively. This proves that NFRP is effective and suitable to enhance the confinement effect of concrete, especially Jute-NFRP.

KEYWORDS

Confinement, natural fibre, Cotton-NFRP, Jute-NFRP, Hemp-NFRP, compressive strength.

INTRODUCTION

During the Mae Lao Earthquake in Thailand (Mw 6.3), many reinforced concrete (RC) structures designed without seismic codes collapsed catastrophically due to insufficient confinement (see Figure 1). One of the most effective methods to increase such confinement effect is to strengthen structures using fibre reinforced polymers (FRPs) because of their light weight, high corrosion resistance and easy installation. Conventional FRP such as Carbon FRP (CFRP) and Glass FRP (GFRP) can significantly increase strength and ductility of structures. However, such FRPs are considered expensive (e.g., 22 US dollars/kg), causing them to be inappropriate for low-cost buildings. As a result, alternative FRPs made of natural fibres such as Natural FRP (NFRP) are necessary for developing countries since they are cheaper and available in local areas (Cook 1968; Cronier et al. 2005; Takasaki et al. 2013; Tara and Jagannatha 2011; Sabbie et al. 2015). Some of the common natural fibres widely produced in Thailand are jute, hemp and cotton fibres for agriculture and textile industries. These fibres are not only economical (0.45 US dollars/kg) but also environmentally friendly. To date, such NFRP has not been widely implemented in engineering applications due to a lack of intensive study on its structural performance.

This paper aims to investigate the tensile behaviour of three types of NFRP coupons, namely Cotton, Jute and Hemp. Therefore, a total of 30 coupon specimens of Cotton, Jute and Hemp NFRP with WARP and WEFT fibre directions was tested based on JSCE-E541 standard (2000). Another goal of this paper is to evaluate the behaviour of concrete confined by those NFRPs and the enhanced compressive strength due to confinement. A total of 63 concrete cylinders with 100 mm-diameter and 200 mm-height was tested and confined by the NFRP with a varying number of layers (1 to 6 layers) and the fibre alignment in only the WEFT direction. Then, all concrete cylinders were loaded under the axial compression test until they failed. It is found that the compressive strength of confined concrete increased significantly up to 28%, 42%, and 25% for Cotton, Jute and Hemp confinement, respectively. The stress-strain relationships of confined concrete showed bi-linear curves, depending on the number of NFRP layer. All NFRP coupons exhibited an approximately bilinear stress-strain relationship (Figure 4b). The average values of the tensile strength of the Hemp-NFRP in WEFT direction was the highest (179 MPa), whereas the
elastic modulus of the Jute-NFRP in WEFT direction was the largest (16,350 MPa). The ultimate strain of the Cotton-NFRP in WARP direction was the largest (5.2%).

**EXPERIMENTAL PROGRAM**

**Tensile coupon test**

To investigate tensile properties of NFRP, a total of 30 coupon specimens of Cotton, Jute and Hemp NFRP with WARP and WEFT fibre directions was tested based on JSCE-E541 standard (2000), as shown in Figure 2. The coupon specimens were denoted with prefix letters “C” for Cotton, “J” for Jute, and “H” for Hemp (see Table 2). All coupon specimens were coated with epoxy resin, and cured for a week. Grips were attached at the both edges of specimens to prevent from a slippage between specimen and the wedge grip of the tensile machine. An image measurement method was used to capture the tensile strain of each NFRP coupon (Figure 2).

**Concrete cylinders and Instrumentation**

A total of 63 concrete cylinders was casted in three batches consisting 1) Group-1 with Cotton-NFRP confinement (denoted as C1 to C6), 2) Group-2 with Jute-NFRP confinement (denoted as J1 to J6), and 3) Group-3 with Hemp-NFRP confinement (denoted as H1 to H6). In each Group, there were 3 control specimens with a diameter of 100 mm and a height of 200 mm (denoted as Control 1 to Control 3 for the 1st batch to the 3rd batch). The compressive strength was designed to be 21 MPa at 28 days, using normal strength cement with coarse aggregate having maximum size of 20 mm. The NFRP only in WEFT direction were wrapped on the concrete’s surface with overlapping zone of quarter of diameter (see Figure 3a). Strain gauges were mounted on the NFRP surface at the mid-height in vertical and horizontal directions. All cylinder concretes were tested under monotonic concentric compression using Universal Testing Machine (see Figure 3b).

**RESULTS AND DISCUSSION**

**Coupon test results**

From the coupon test, tensile strength, elastic modulus and rupture strain of 30 coupon specimens were obtained. Table 2 summarizes the average tensile properties of all three types of NFRP composites, including their elastic modulus, rupture strain, and tensile strength. There are two values of elastic modulus, namely the initial elastic modulus ($E_1$) for the 1st region and the 2nd stage elastic modulus ($E_2$) for the 2nd region. The NFRP composites failed in either mid-height or end-grip of the specimens (Figure 4a). All NFRP coupons exhibited an
approximately bilinear stress-strain relationships, considering the average values of each NFRP coupons (Figure 4b). The average values of the tensile strength of the Hemp-NFRP in WEFT direction was the highest (179 MPa), whereas the elastic modulus of the Jute-NFRP in WEFT direction was the largest (16,350 MPa). The ultimate strain of the Cotton-NFRP in WARP direction was the largest (5.2%).

Table 2 Tensile properties of coupon specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Thickness (mm)</th>
<th>Average tensile Strength (MPa)</th>
<th>Average Initial Elastic Modulus $E_1$ (MPa)</th>
<th>Average 2nd stage Elastic Modulus $E_2$ (MPa)</th>
<th>Average Ultimate Strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-WARP</td>
<td>0.31</td>
<td>74</td>
<td>5,145</td>
<td>443</td>
<td>5.2</td>
</tr>
<tr>
<td>C-WEFT</td>
<td>0.31</td>
<td>104</td>
<td>6,446</td>
<td>1,339</td>
<td>4.0</td>
</tr>
<tr>
<td>J-WARP</td>
<td>0.14</td>
<td>155</td>
<td>9,409</td>
<td>4,978</td>
<td>2.2</td>
</tr>
<tr>
<td>J-WEFT</td>
<td>0.14</td>
<td>174</td>
<td>16,350</td>
<td>11,834</td>
<td>1.3</td>
</tr>
<tr>
<td>H-WARP</td>
<td>0.18</td>
<td>144</td>
<td>3,293</td>
<td>2,092</td>
<td>5.3</td>
</tr>
<tr>
<td>H-WEFT</td>
<td>0.18</td>
<td>179</td>
<td>11,631</td>
<td>3,690</td>
<td>2.5</td>
</tr>
</tbody>
</table>

Figure 4 Failure modes and average stress-strain relationships of cotton, jute, and hemp NFRP coupon specimens

Figure 5 Failure modes of confined concrete cylinders
Concrete cylinders

Failure modes

Failure modes of confined concrete cylinders tested under monotonic loading are shown in Figure 5. For the first batch of specimens, all the NFRP-confined concretes failed by tensile rupture of the NFRP with a loud noise at the ultimate state. In some Cotton-NFRP and Jute-NFRP confined concretes, slippage between NFRP layers can be observed (see C5, C6, J5 and J6 in Figures 5a and 5b).

Stress-strain relationships

Figure 6 presents the compressive stress-strain curves in both lateral and axial directions for all unconfined and confined concrete cylinders. Both the compressive strength and the ultimate axial strain were significantly enhanced in confined concrete cylinders. Overall stress-strain relationships of NFRP confined concrete cylinders exhibit a bilinear shape. In the 1st region, the stress–strain behaviour of NFRP confined concrete cylinders is slightly similar to the corresponding unconfined ones. In the 2nd region, the curve enters the nonlinear transition region where considerable micro-cracks are propagated in concrete and the lateral expansion significantly increased. The NFRP then starts to confine the concrete core, leading to an increase in compressive strength and ductility of concrete. However, such enhancement can develop at a certain level, depending on the number of NFRP layer. With lower amount of NFRP, confined concrete could exhibit a softening behaviour. On the other hand, they could exhibit an ascending behaviour with higher amount of NFRP. The lateral tensile stress in the NFRP also increases when axial stress increases. Once this lateral stress exceeds the ultimate tensile strength of NFRP obtained from the flat coupon tensile test failure of the NFRP tube starts.

Figure 6 Stress-strain relationships of NFRP confined concrete

Figure 6a shows the stress-strain behaviour of Cotton-NFRP confined concrete cylinders. When applying 1 to 5 layers of Cotton-NFRP, the stress-strain curve after the peak strength shows a rapid softening (e.g. C1-C5) owing to the low confinement. When applying Cotton-NFRP to 6 layers (C6), the confinement increases so that the
stress-strain curve after the peak exhibits an ascending linear behaviour until the fracture of NFRP (see Figure 6a). For Jute-NFRP confined concrete, the stress-strain behaviour also shows significant enhance of compressive strength (see Figure 6b). The softening region after peak strength can be observed in J1-J4, whereas the ascending region after the peak strength can be seen in J5-J6. However, the Hemp-NFRP confined concrete (H1-H6) exhibit only softening behaviour after the peak strength because of a small tensile stiffness $E_{frp} (1.63 \text{ GPa-mm})$ than Cotton-NFRP (2.00 GPa-mm) and Jute-NFRP (2.94 GPa-mm) and premature fracture, leading to insufficient confinement effect.

Overall, the increment in compressive strength of concrete increase significantly up to 28%, 42%, and 25% for Cotton, Jute and Hemp confinement, respectively. It can also be observed that the lateral rupture strain of NFRP jackets measured in compression tests was remarkably lower than the ultimate tensile strain of NFRP coupon specimens. According to Lam and Teng (2004) and Dai et al. (2011), such difference was owing to the deformation non-uniformity of cracked concrete.

Table 3 Comparison between the compressive strength from test and model

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$f'_{cc}$ - test (MPa)</th>
<th>$f'_{cc}$ - model (MPa)</th>
<th>Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control 1</td>
<td>17.7</td>
<td>17.7</td>
<td>0</td>
</tr>
<tr>
<td>C1</td>
<td>19.1</td>
<td>24.1</td>
<td>26.3</td>
</tr>
<tr>
<td>C2</td>
<td>19.3</td>
<td>29.6</td>
<td>53.2</td>
</tr>
<tr>
<td>C3</td>
<td>19.6</td>
<td>35.1</td>
<td>79.4</td>
</tr>
<tr>
<td>C4</td>
<td>20.3</td>
<td>40.6</td>
<td>100.2</td>
</tr>
<tr>
<td>C5</td>
<td>20.5</td>
<td>46.1</td>
<td>124.6</td>
</tr>
<tr>
<td>C6</td>
<td>22.7</td>
<td>51.6</td>
<td>127.6</td>
</tr>
<tr>
<td>Control 2</td>
<td>21.5</td>
<td>21.5</td>
<td>0</td>
</tr>
<tr>
<td>H1</td>
<td>22.2</td>
<td>24.3</td>
<td>9.5</td>
</tr>
<tr>
<td>H2</td>
<td>24.6</td>
<td>27.1</td>
<td>10.4</td>
</tr>
<tr>
<td>H3</td>
<td>24.8</td>
<td>30</td>
<td>20.7</td>
</tr>
<tr>
<td>H4</td>
<td>25.1</td>
<td>32.8</td>
<td>30.8</td>
</tr>
<tr>
<td>H5</td>
<td>25.3</td>
<td>35.6</td>
<td>40.8</td>
</tr>
<tr>
<td>H6</td>
<td>27</td>
<td>38.5</td>
<td>42.5</td>
</tr>
<tr>
<td>Control 3</td>
<td>20.1</td>
<td>20.1</td>
<td>0</td>
</tr>
<tr>
<td>J1</td>
<td>20.8</td>
<td>22.7</td>
<td>9.3</td>
</tr>
<tr>
<td>J2</td>
<td>22.2</td>
<td>25.3</td>
<td>14.1</td>
</tr>
<tr>
<td>J3</td>
<td>22.7</td>
<td>28</td>
<td>23.4</td>
</tr>
<tr>
<td>J4</td>
<td>23.2</td>
<td>30.6</td>
<td>32.2</td>
</tr>
<tr>
<td>J5</td>
<td>25.1</td>
<td>33.3</td>
<td>32.7</td>
</tr>
<tr>
<td>J6</td>
<td>28.7</td>
<td>35.9</td>
<td>25.3</td>
</tr>
</tbody>
</table>

Figure 7 presents the comparison between the compressive strength from test and model. Using a linear regression, it is concluded that the compressive strength predicted by current existing model is unconservative, which is approximately 1.42 times of the tested values (see Figure 7). Therefore, the modified equations for several types of NFRP are crucially required in the future research program.

Modelling of compressive strength

For confined concrete design, the ultimate axial compressive strength is one of the significant parameters for engineering design. According to Dai et al. (2011), the compressive strength model was proposed based on Teng and Lam’s model (2003). Dai et al. (2011) proposed the model which is appropriate for both conventional FRPs (e.g., CFRP and AFRP) and large rupture strain FRP (LRS-FRP) such as PET-FRP and PEN-FRP, as shown below:

$$ f'_{cc} = f'_{cu} + 3.5\sigma_l $$

where $f'_{cc}$ = confined compressive strength; $f'_{cu}$ = unconfined compressive strength; and $\sigma_l$ = confining pressure (or lateral stress) of the FRP jacket.
\[
\sigma_i = \frac{\sigma_{b,\text{rup}} t_{frp}}{R}
\]

where \(\sigma_{b,\text{rup}}\) = hoop stress of FRP jacket \((= E_{frp} \varepsilon_i)\); \(t_{frp}\) = nominal thickness of the FRP jacket; and \(R\) = radius of the concrete core; \(E_{frp}\) = secant modulus of NFRP obtained from coupon test; \(\varepsilon_i\) = hoop or lateral strain of NFRP at peak strength obtained from coupon test.

Table 3 shows a comparison between the predicted and the experimental results for the NFRP-confined concretes. The compressive strength equations proposed by Dai et al. (2011) can successfully estimate the strength in the concrete specimens confined with a small amount of NFRP confinement, namely 1 or 2 layers (with percent error of 9.3% to 26.3%). Meanwhile, the model overestimates compressive strength of confined concrete with a larger amount of NFRP, especially C4-C6 (with percent error of 100.2% to 127.6%). This is because the model was generated based on an interpretation of the existing test data of conventional FRP, PEN-FRP and PET-FRP confined concrete.

CONCLUSIONS

This study experimentally investigated the compressive behaviour of Cotton, Jute, and Hemp NFRP confined concrete cylinders (wrapped only in the WEFT direction). The study can be summarized as follows:

1) All NFRP coupons exhibited an approximately bilinear stress-strain relationship in terms of average values. The average values of the tensile strength of the Hemp-NFRP in WEFT direction was the highest (179 MPa), whereas the elastic modulus of the Jute-NFRP in WEFT direction was the largest (16,350 MPa). The ultimate strain of the Cotton-NFRP in WARP direction was the largest (5.2%).

2) In axial compression, NFRP confinement significantly enhances the compressive strength and ductility for confined concrete. The enhancement in strength and ductility increases with an increase in amount of NFRP. The compressive strength of confined concrete increases significantly up to 28%, 42%, and 25% for Cotton, Jute and Hemp confinement, respectively.

3) Cotton-NFRP (with 1-5 layers) and Jute-NFRP (with 1-4 layers) confined concretes under compression exhibit bi-linear stress-strain behaviour with a linear-ascending branch at the first stage, following by softening branch at the second region. When increasing Cotton-NFRP to 6 layers and Jute-NFRP to 5-6 layers, the second regions show ascending behaviour owing to sufficient confinement level. However, only softening behaviour in the second region was observed in Hemp-NFRP confined concrete because of its low elastic modulus and premature fracture.

4) Dai et al.’s model can successfully predict the ultimate compressive strength of NFRP confined cylinders with lower amount of NFRP layer (1 to 2 layers). However, the model is unable to predict the ultimate strength of confined concretes with higher amount of NFRP. The compressive strength predicted by current existing model is unconservative, which is approximately 1.42 times of the tested values. The modified equations for several types of NFRP will be proposed in the future research program.

ACKNOWLEDGMENTS

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