FRP INTERNATIONAL

The Official Newsletter of the International Institute for FRP in Construction

Volume 2, Issue 1 ***February 2005***

This issue finds IIFC as a growing and vibrant organization, as exemplified most recently by the tremendous success of the 2nd International Conference on FRP Composites in Civil Engineering (CICE 2004) held in Adelaide, Australia in December 2004. Kudos go to Dr. Rudi Seracino, Chairman of the CICE Organizing Committee, his team, and the many participants. We will be focusing on an overview of the activities of this conference and the discussions held at the concurrent IIFC Council meetings in a forthcoming issue of the Newsletter.

A newsletter is only as successful as the level of interest and participation of the members of IIFC and the FRP community at large. As we move forward I'd like to invite our readers to submit material to the newsletter on new applications of FRP in Construction, forthcoming conferences and workshops, or even general items that may be of interest to the worldwide community. Ongoing research, code development, and implementation activities are of great interest as well. Material can be submitted directly to me at vkarbhari@ucsd.edu.

Please also feel free to write to me or to the President of IIFC, Prof. J.G. Teng (cejgteng@polyu.edu.hk), with any ideas you may have for the newsletter and for IIFC, itself.

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In this issue

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Volume 2, Issue 1 **February 2005**

Reports From Around the World

In this issue we highlight applications related to the use of FRP composites in bridge decks and girders.

The "Manitoba" GFRP Bridge Deck System

by
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With an estimated 80,000 Canadian bridges and 230,000 U.S. bridges in need of serious repair (Magazine of Popular Science, 1998), emphasis is being placed on designing and building bridges that will last longer, while requiring minimal maintenance. Steel reinforcement and structural steel members are known to be susceptible to corrosion, while concrete could also crack and spall due to sulfate attack, freeze-thaw and other detrimental processes. The combination of material degradation and substandard load ratings has led many bridge structures to be classified as structurally deficient or functionally obsolete. At this point, repair or replacement is required and this cost can reach as high as 75% to 90% of the total annual maintenance cost of the structure (Karbhari et al, 2001). When repair or replacement is imminent, there is not only the associated cost of materials and labour, but also the cost of losses due to delays and detours. In order to provide effective alternatives, this paper proposes the use of an innovative design for bridge deck system using Fiber Reinforced Polymer (FRP) materials.

A modular GFRP bridge deck was developed in Manitoba, Canada as a feasible alternative to conventional construction in the repair and replacement of North America's aging highway bridges. An experimental program was undertaken at ISIS Canada at the University of Manitoba to study the behavior of the GFRP bridge deck patented by Wardrop Engineering Inc., Winnipeg, Manitoba and Faroex Ltd. Composites, Gimili, Manitoba (US patent No.6,151,743). The research and development project described was founded on the evolution of four generations of bridge deck modules. Based on the behavior observed by the first and second generations, third generation deck was fabricated and tested. Testing of the first two generations was performed on deck modules using filament-wound triangular tubes, with and without outer plates. From these results, the behavior of the various deck components was investigated (Williams *et al.*, 2003). The third generation deck investigated the effects of filament-winding the entire section to produce components that would resist bending. The fourth generation deck was produced using both filament-wound tubes and pultruded laminates with the optimum amount of fibers, resulting in the most cost-effective deck of all four generations. The GFRP deck is designed to support the HS30 design truck wheel load plus dynamic load allowance (DLA), while being less than 25 pounds per square foot in weight.

Static tests were performed on all generations, with the exception of the third generation in which cyclic loading was performed to determine the effect of fatigue. The module, shown in Fig. 1, was fatigued to 2,000,000 cycles with a load varying between 10% and 135% of the service load, and finally tested to failure.

Fig. 1: Test Setup of GFRP Deck Module

The performance was evaluated based on load capacity, mode of failure, deflection at service load level, strain behavior, and stiffness degradation under cyclic loading. All decks tested exceeded the requirements to support the HS30 design truck wheel load of 140 kN including DLA. Fig. 2 shows the load deflection diagram for the final deck design.

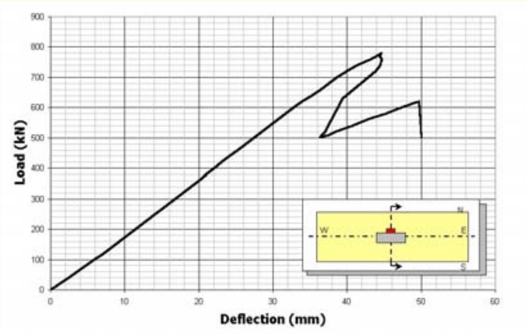

Fig. 2: Load-Deflection Diagram for Fourth-generation GFRP Deck Module

The GFRP deck loaded using a single load at the midspan exhibited linear behaviour up to failure. The deck failed at a load of 780 kN, that is six times the service load level. The failure was initiated by localized delamination buckling while the deck showed sufficient deformation prior to failure. Because of the relatively low stiffness of the FRP material, the deck design is controlled by the serviceability limit state. The deflection at service load level was selected not to exceed span/425 as recommended in the AASHTO LRFD for timber decks to limit possible cracking in the wearing surface.

Upon development of the bridge deck system, the need arose to develop a means of connecting adjacent bridge deck panels. A hollow triangular filament wound shear key was selected as a solution to the design challenge. A full-scale prototype deck was manufactured and tested (Fig. 3). The shear key connection was subjected to two million cycles of an equivalent AASHTO HS-30 wheel load plus DLA to determine the effect of repeated loading on its stiffness and overall performance. Finally, the deck was tested statically to failure to determine the postfatigue failure strength of the specimen and to examine the failure mode of the deck.

Fig 3: Test Setup of Shear-Keyed Deck Module

To be adequate, the shear key should demonstrate minimal degradation of stiffness after repeated loading. It should also transmit load between adjacent deck modules with minimal differential movement between the adjacent panels. At the completion of two million load cycles, the jointed deck demonstrated only a 3% loss in stiffness The load-deflection behaviour of the deck was linear throughout the testing program. After two million load cycles the service load deflection of the specimen was 6.1 mm, which corresponds to L/480. Testing of a deck specimen without the shear key showed a midspan deflection of 7.1 mm (L/425) at the service load level (Fig. 3), therefore, the insertion of the shear key enhances the overall stiffness of the deck by 14%. Consideration of the deflection profile across the deck at midspan demonstrates that minor differential displacement occurred at the joint. The differential displacement measured on either side of the shear key at service load level (140 kN) varied between 0.15 and 0.25 mm throughout the two million load cycles. These values correspond to less than 4% of the specimen's total midspan deflection. Therefore, the shear key transmits the load efficiently between adjacent deck modules.

Wardrop Engineering and ISIS Canada are working with the Province of Manitoba Department of Transportation to design and instrument a movable bridge with FRP deck system that will be used for the Northern Manitoba winter roads.

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FRP Bridge Decks and Systems for Road Bridges - European Perspective

by

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Compared to development in the United States, the use of Fiber-Reinforced Polymers (FRP) in bridge deck or bridge superstructure applications for road bridges is in its infancy in Europe. Until now, the use of FRP composites in new constructions has been focused on pedestrian bridges. A multitude of all-FRP or hybrid-FRP pedestrian bridges have been built in several European countries (Keller, 2003). Permanent road bridges with FRP decks or superstructures, however, have only been built in the UK. In Germany, an emergency bridge for temporary use has been developed. In Switzerland, an extensive research project on the use of FRP bridge decks is ongoing. The activities in these three countries are summarized in the following.

United Kingdom

In the UK the ACCS system (Advanced Composites Construction System) was developed by Maunsell Structural Plastics in the late 1980s. The system is composed of three pultruded profile types: multi-cellular planks, square tubes and toggle-bars to form interlocking adhesively bonded joints. These basic elements enable the fabrication of large square and rectangular box sections (cf. Fig. 1, Lee et al., 1995).

Fig. 1: GFRP box beam in construction using ACCS planks

The system was originally developed for protective bridge enclosures and was first used in 1988 for the A19 Tees Viaduct. Later, the system was used for superstructures of pedestrian bridges (e.g. Aberfeldy Bridge, 1992) and for the Bonds Mill Lift Bridge in 1994. For this road bridge, however, it was necessary to stabilize the cell walls of the deck with stacked planks and foam filling (Turvey, 2001). A new bridge deck plank, whose design is based on the ACCS system is under development. The new six-cell roadway panel has a similar geometry to the lighter ACCS plank, but with thicker sections to resist direct wheel loads. It was designed to be laid transversely on a sub-structure built from ACCS planks (Daly and Cuninghame, 2004). Based on an extensive test program on this combined system, the preparation of a design guideline for FRP bridge decks is under way (Sadka and Daly, 2003).

As a result of a four-year research project funded by the European Union, the ASSET bridge deck system was developed and implemented in 2002 for the West Mill Bridge in Oxfordshire. The ASSET deck system is composed of pultruded GFRP two-cell sections with a triangular cell form, which are adhesively connected to form the deck (cf. Fig. 2).

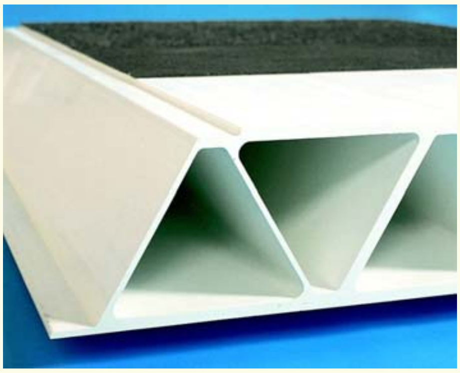

Fig. 2: ASSET bridge deck system

The ASSET section, fabricated by Fiberline Composites, Denmark, was developed for a 2 m span and 40 t load, based on HA and HB loading as described in the UK code BS5400 (Luke et al., 2002). The West Mill Bridge has a span of 10 m and a width of 6.8 m for two lanes (cf. Fig. 3).

Fig. 3: GFRP West Mill Bridge in construction using ASSET system

The superstructure consists of four longitudinal beams with the ASSET deck spanning these beams in the transverse direction. In addition, reinforced concrete parapet edge beams hold the parapets in place and in turn are anchored to the deck. Each of the longitudinal beams is composed of four adhesively bonded pultruded GFRP box sections with uni-directional CFRP flanges to provide the required global flexural rigidity. The deck is adhesively bonded to the longitudinal beams and is covered by a polymer concrete and epoxy-based wearing surface. The bridge is extensively instrumented for long-term monitoring. Field testing after installation showed a large degree of composite action between the main beams, the ASSET deck and the reinforced concrete parapets. The presence of composite action doubled the short-term global flexural rigidity of the bridge (Canning and Luke, 2004).

Germany

In 2001, the Institute of Steel Construction and Lightweight Structures at RWTH Aachen, Germany constructed an emergency single-lane road-bridge. The 20 m span bridge is 2.75 m wide and is composed of pultruded GFRP sections of quality E23 according to the European Standard EN13706 (2002). The sections form two truss girders arranged laterally above the deck. The truss joints are bolted and reinforced by adhesively bonded steel plates. The deck consists of GFRP plank profiles supported by cross-beams. The bridge can be erected by hand by six unskilled workers in three hours (Sedlacek and Trumpf, 2004).

Switzerland

From the approximately 3000 bridges on the national highway system in Switzerland, 40 to 50 bridges per year will require replacement in the future. Their replacement must be rapid and should not interfere with traffic to avoid an adverse effect on the economy and becoming politically unacceptable. The use of FRP decks - first as part of temporary bridges for traffic diversion and later as part of permanent bridges with small to mid-range spans - is an attractive solution. For this reason, the Swiss Federal Roads Authority awarded the Swiss Federal Institute of Technology in Lausanne a three-year research project to develop appropriate temporary and permanent bridge systems using FRP decks for spans up to 50 m and widths up to 11 m for three lanes of traffic. At the beginning of the project, three fundamental conceptual design decisions were made: (1) steel girders are to be used for the main girders, (2) the FRP decks are to be adhesively bonded to the steel girders, (3) the deck slab is to participate fully in the longitudinal bridge direction (full composite action). Furthermore, the bridge systems are to be supported by design guidelines as well as recommendations on construction detailing.

The research project was divided into three stages. In the first project stage, the in-plane load-carrying performance of FRP decks acting compositely as part of the top chords of steel girders was examined. Keller and Gürtler (2004) proposed in-plane system properties for two pultruded FRP bridge deck systems (DuraSpan and ASSET), which comprise the material properties and the effects of cell geometry and adhesive joints between the pultruded shapes.

In the second project stage, the static and fatigue behavior of full-scale girders subjected to four-point bending was examined, as shown in Fig. 4.

Fig. 4: DuraSpan system adhesively bonded to steel girders (test girders at onset of failure)

Each of the girders consisted of a welded steel girder and an adhesively connected pultruded FRP bridge deck panel that acted as part of the top chord. DuraSpan and ASSET deck panels were used. The layer thickness of the two-component epoxy adhesive used was between 6 mm and 10 mm. In all FRP-steel girders failure occurred in the compressed deck during tensile yielding of the bottom part of the steel girder, with full composite action in the adhesive layer up to failure. The adhesive deck-to-girder connections remained undamaged. The girders showed no damage after ten million fatigue load cycles and no sensitivity to creep deformations.

In the third project stage, analytical methods to predict stresses and deflections at the serviceability limit state and ultimate failure loads were developed. It was found that the adhesive bond always provides full composite action between the FRP decks and steel girders, even in the case of flexible polyurethane adhesives with a layer thickness up to 50 mm. For pultruded bridge deck systems, however, the webs or diagonals between the upper and lower face panels provide only partial shear connection of the upper face panels in the longitudinal bridge direction.

Based on the results of this extensive research project, a temporary bridge with an FRP deck adhesively bonded to steel girders will be built in Switzerland in 2005. The single-lane two-span bridge (2x20 m) will carry up to 11000 vehicles per day for two years, with a high porportion of heavy truck traffic. The static, dynamic and fatigue behavior of the bridge will be studied.

In a parallel project, the fire endurance of the DuraSpan bridge deck system was investigated. Deck elements were subjected to serviceability limit state loads during ISO 834 fire exposure from the underside. The full-scale specimen provided structural resistance during fire exposure for 57 minutes. It was concluded that the fire endurance of cellular GFRP bridge decks is sufficiently high in the case of fire from the underside.

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Research on FRP deck in Japan

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The use of FRP deck in Japan has lagged behind its introduction in Europe and America. Between 30 and 40 years have passed since expressways, mainly urban type elevated roads, were first constructed, and fatigue of the concrete and steel decks has begun to become a problem. Standards have been revised so that their bearing load could be increased from 20 tons to 25 tons. For these reasons, many studies of the application of FRP as a method of reinforcing existing decks have been carried out. But decks made only of FRP (not the reinforcement of concrete decks using FRP) discussed in this report have almost never been used in Japan. In Japan, spike tires have been prohibited in recent years, increasing the use of deicing salts, but because not nearly as much is spread as in Europe and America, there is now almost no need for the application of FRP decks as a way to prevent corrosion. Present technology, in other words, deck waterproofing materials and cathodic protection, can generally deal with this problem. The major reason for performing corrosion protection in Japan is to prevent corrosion caused by salinity along sea coasts: an environments where bridge girders and piers are more susceptible to its effects than decks. For these reasons, decks made only of FRP have been used only rarely and detailed studies have not been done. A composite deck with concrete and FRP is already developed in Japan and applied to some bridges (Maeda, 2003), but this technology is not discussed in this report because it's not made only of FRP. This report introduces two major research examples: (1) a study by the PWRI and its Test Emergency Bridge and (2) an all-FRP bridge (the Okinawa Road-Park Bridge).

1. Example of a study conducted by the PWRI and its application to the Test Emergency Bridge of the PWRI (PWRI, 2000)

The basic structure of this deck is general purpose square pipe type pultruded GFRP lined up laterally and connected with bond, then lightly prestressed with PC steel bars. As a preliminary test, static loading properties of individual square pipes were tested. It confirmed that interlaminar peeling of the FRP occurred, resulting in failure in the web directly below the load point. The stress directly below the load point at failure time was analyzed by linear FEM, revealing that the stress concentration caused the shear failure of the corner of web and flange. Similar loading tests were performed using specimens prepared by filling the inside of the square pipes with light weight mortar. Failure caused by stress concentration directly below the load point was not observed. The results of linear FEM analysis of the interlaminar shear stress distribution of the web of the FRP beam filled with mortar at the time of failure confirmed that its shear stress was lower than that without a mortar filling.

In another test, it was also confirmed that a pultruded GFRP pipe available on the market, even though it was filled up with mortar, undergoes brittle failure caused by horizontal shear failure of the FRP member. This failure is believed to be a result of insufficient reinforcement at right angles to the member axis. A static loading test of the quantity of reinforcing fibers in the direction at right angles to the member axis of an FRP beam performed using a square pipe reinforced with the use of bias knit fabric, obtained an FRP beam member that undergoes compression failure and tensile failure, but not brittle failure.

The deck specimens were prepared by first lining up 24 of the above square pipes and bonding the FRP to each other with epoxy resin, then using 2 PC steel bars to introduce prestressing of 3MPa to the joint surfaces. Three specimens, one without mortar, one with normal mortar, and one filled with super lightweight mortar, were prepared. The loading was static loading on the center of the deck using a loading plate with diameter of 50mm (Fig. 1).

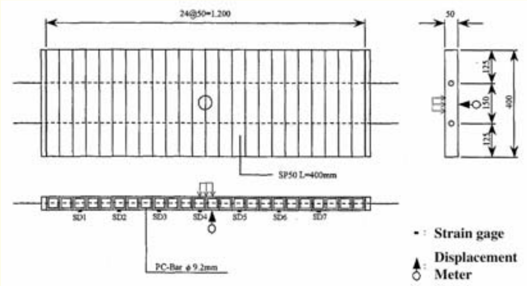

Fig. 1: Outline of FRP deck specimen (unit: mm)

All specimens underwent punching shear failure directly under the loading point without any mutual displacement of the pipes, but it was confirmed that there were differences of about four times in the failure load. It was confirmed that in the specimen containing no mortar, the web suffered progressive failure only near the point directly below the loading point as it was crushed and buckled, but in the specimens filled with mortar, only strain distributed in the bridge axis direction was produced.

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In order to study the applicability of an FRP deck made of square FRP pipes for use as a bridge deck, the wheel loading test was done on the specimens shown in Figure 1. The wheel loading test was done as shown in Figure 2; applying a moving load at a cycle of 0.25Hz as the loading wheels applied load of 10kN. During the test, the moving speed was lowered to perform wheel loading at intervals from a few tens of loading cycles to a few hundred loading cycles.

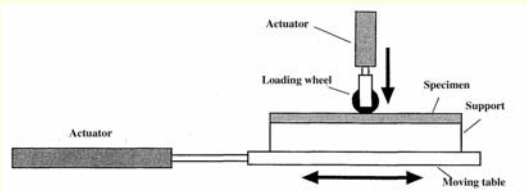

Fig. 2: Outline of the Wheel Loading Test

It confirmed that in the specimens without mortar, the top flange was cracked longitudinally by the first loading, and that at the 53rd loading cycle, in all the flanges of the part where the wheel passed, longitudinal cracking of the center of the span, and the joint of the flange and web occurred, causing the wheel to sink into the deck. Studies of the FRP deck performed in this way have revealed that it is necessary to reinforce it against local strain, and that an effective way to do this is to reinforce the deck by filling hollow FRP with relatively inexpensive mortar.

This deck was used to construct an experimental emergency road bridge at the Asagiri Test Site of the Public Works Research Institute in 1998, however only FRP deck without mortar was used to the deck. (Fig. 3) The purpose of this experimental bridge was developed for "easy erection" on disaster such as earthquake.

Fig. 3: Test Emergency Bridge of PWRI

2. The study of the Okinawa Road Park bridge (Kitayama and Uno, 2001)

The Okinawa Road Park Bridge constructed in 2001 is Japan's first FRP pedestrian bridge. As shown in Figure 4, it is a 2-span continuous girder bridge with total length of 37.76m and width of 3.5m. This bridge is made of FRP decks. In order to minimize its cost, hollow panels made of pultruded FRP with existing section were used (Fig. 5). A shear key was installed at right angles to the bridge axis. Treating the bridge axis direction as the deck span direction, floor systems were installed at intervals of 1.5m, to construct its spans as simple girders. The design load is 4,900N/m^2 based on Japan's pedestrian bridge standards.

Fig. 4: The Pedestrian Bridge in the Road-Park of Ikei-Tairagawa in Okinawa

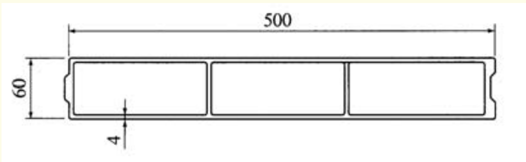

Fig. 5: FRP deck plate panel (unit: mm)

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Alternative Hardwood Bridge Girders – An innovation with composites

by
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Australian hardwoods are an excellent general purpose building material, however in recent years they are becoming more expensive, less available, and of poorer general quality than has previously been the case. Many Australian timber bridges will remain in service for the foreseeable future, and the maintenance and potential upgrading of these structures will be an on-going demand, while the availability of traditional resources declines. Compatible alternative timber bridge components are therefore required. FCDD has been developing hybrid composite/timber beams for several years in collaboration with major timber bridge asset owners, and is now in the process of specialising products to meet the needs of specific asset owners. This paper summarises recent developments, and discusses some of the important issues to be considered.

Australian hardwoods are an excellent general purpose building material. This was recognised by early road builders in Australia, and hardwood timber bridges proliferated, particularly during the first half of the 20th century. Timber used in these structures was typically the best of the old growth forests. Australia now has a large number of timber bridge structures (possibly as many as 20,000) that are in the high maintenance phase of their useful life. Many bridge asset owners are confronted with the following realities regarding these timber structures:

1. There are insufficient funds available to construct replacement structures;
2. The assets must remain in a safe usable condition;
3. There is a growing shortage of large section hardwood suitable to repair and rehabilitate these structures.

This situation has created a need for effective alternatives.

While the shortage of hardwood timber bridge components is relatively easy to define as a problem, development of solutions to this problem requires a relatively detailed understanding of a range of issues. Timber bridge technology has developed with a focus on workable solutions rather than performance specifications. Ascribing performance specifications to timber bridge components that are generic enough for other materials to be considered as alternatives is a challenge. This paper begins with a discussion of these issues based on the recent experience of Fibre Composite Design and Development (FCDD). One of FCDD's hardwood substitute alternatives is then described, along with a summary of development of this product to-date.

Characteristics of traditional hardwood timbers are relatively well understood (at least implicitly) by the bridge engineering community in Australia, even if this does not extend to a codified approach to the engineering of timber bridges. They are considered strong, durable, reliable, easy to work with, somewhat variable, and (until recently) readily available. There is a strong empirical knowledge base with respect to safe and workable solutions, and the material is considered to be relatively “forgiving”. Some of these characteristics are relatively easy to quantify, in particular strength, stiffness and cost, whilst others are quite difficult to quantify. For example hardwood is considered to be easy to “machine”. This is well understood in the context of hardwood practitioners, but relatively difficult to define in terms of performance parameters. Ease of machining is considered to be a desirable characteristic, and is essential given that there is considerable dimensional variability in traditional hardwood girders supplied to road authorities. Consequently, development of a competitive alternative to hardwood girders will require close collaboration between product developers, asset owners and bridge gangs, particularly with respect to work practices.

The philosophy of the hybrid beam concept developed by FCDD is based on the optimal use of different materials. The concept uses plantation softwood in the form of plywood or laminated veneer lumber (LVL) for the bulk of the beam, with composite reinforcement modules to increase the strength and stiffness to a level equivalent to that of high quality hardwood. The timber is used to provide the shear capacity for the beam, maintaining the separation between the reinforcement modules, and to provide the functionality associated with timber.

The concept of a reinforced timber beam, utilising relatively low-performance timber as a core, and other higher performance materials providing additional stiffness and strength is not new. The main novelty involved in the approach used by FCDD relates to the combination of materials used to form the reinforcement.

The reinforcement modules use a combination of composite materials and have a Modulus of Elasticity of 60GPa and a failure strength of around 200MPa. The modules are bonded to the timber using a high strength epoxy adhesive. The stress in the adhesive is relatively low due to the large surface area of the modules. The hybrid nature of this reinforcing system provides a great degree of flexibility to engineer specific properties into the end product.

Considerable development was necessary to advance this concept towards a pre-engineered alternative hardwood girder. For the purpose of comparison, two beams with a constant cross section 186 wide x 200 deep were manufactured using ACQ treated plywood timber. The first was made from solid F14 plywood. The second beam was similar however four composite modules were added to it (Figure 1).

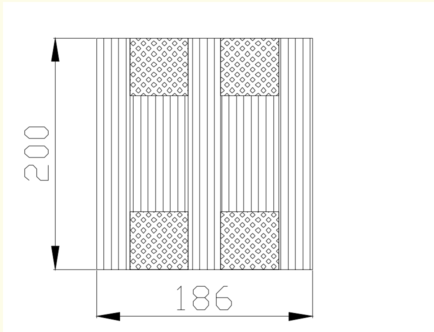

Fig. 1: Cross-Section of a 3m Test Beam

Figure 2 shows the test results of initial small scale experiments that were undertaken to investigate the behaviour of the hybrid beam. The first (plywood) beam closely matched the characteristic stiffness of the Australian timber code, but had an ultimate strength 40% higher than the characteristic bending strength of 20MPa (f'_b in AS1720.1-1997 Section 5 is 40MPa, with 50% of plies having the grain direction parallel to the span). The results for this test are shown in Fig. 2. Typical code results for F22 (Characteristic flexural strength 80MPa, Average MoE 18.5GPa) and F34 (Characteristic flexural strength 100MPa, Average MoE 21.5 GPa) timber are also given in Fig. 2.

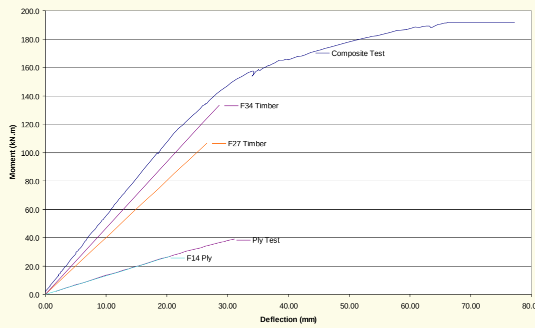

Fig. 2: Moment-Deflection Results

The second (composite reinforced) beam had a stiffness greater than the characteristic stiffness of an equivalent F34 timber beam, and a strength 44% higher than the characteristic strength of

F34. The composite hybrid beam also had a pseudo ductile failure mode, providing significant warning of failure through cracking of the ply and the associated large deflections.

Prototype girders have already been delivered to the Roads and Traffic Authority of NSW (RTA) and the Rail Infrastructure Corporation of NSW (RIC), and plans are well advanced to deliver prototype girders to the Queensland Department of Main Roads (QDMR). The following pictures show examples of 18 and 20m long hybrid beams for use in road-over-rail bridges which require shallow abutment depth and a flat soffit, to minimise earthworks and retain minimum track clearances.

Fig 3: 18m Bridge Girders Before Shipping to Site

Fig. 4: 18 m Bridge Girder Installed

Fig. 5: Load Testing

Fig. 6: 20m Long Hybrid Beams Ready to be Shipped to Site

Work to date indicates that the hybrid beam concept provides a viable alternative to high quality hardwoods which have traditionally been obtained through unsustainable logging of mature native forests. In addition to the more sustainable raw materials associated with this new concept, the beam only incorporates one-sixth the embodied energy in its manufacture compared to conventional steel and concrete beams while providing superior load carrying capacity at an equivalent cost. The beam also has a positive environmental impact in terms of greenhouse gas generation, as it stores more CO_2 than it releases.

The decreasing access to hardwood timber both in terms of volume and quality, has created strong demand in Australia for alternative solutions from asset owners with large inventories of hardwood timber bridges. FCDD has been developing hybrid composite/timber beams for several years, and is in the process of commercialising a range of products to meet the needs of specific markets.

It has been demonstrated that economical viable alternatives can be produced that match and in some cases outperform the structural behaviour of old growth hardwood bridge girders. There is much to be gained for all concerned by exploring these new possibilities, and this requires collaboration between asset owners, product developers, bridge maintenance staff, manufacturers, and indeed the community.

A Perspective From Industry

In each issue the newsletter will highlight a perspective or report from industry. In this issue Matthew Samms of Martin Marietta Composites, discusses the use of FRP decks on the historic Broadway Bridge in Oregon. Besides being a wonderful example of the efficacy of use of FRP composites the bridge is also the 7th longest bascule bridge in the world.

Broadway Bridge: An Ideal Application for FRP Bridge Decks

by
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It is not uncommon to combine the benefits of multiple elements to address our increasingly complex demands. In ancient times, craftsmen constructed huts of mud reinforced with straw. Today, contractors construct high-rise buildings of concrete reinforced with steel. In both cases a matrix (mud or concrete) encapsulates and protects load-bearing fibers (straw or steel) (1). These "composite" materials combine two primary constituent elements to accomplish a singular task – carry loads in an efficient, durable manner. Fiber-reinforced polymer (FRP) materials are another example of a composite solution to today's complex demands. FRP materials combine a polymer matrix (resin, additives, and fillers) with a reinforcing agent (glass, carbon, etc). The marine and aerospace industries have utilized FRP materials for several decades (1). FRP's combination of high strength, low weight, and corrosion resistance often position it as a singular solution to designers' challenges. Its ability to be customized for part reduction provides designers with a wealth of freedom. In spite of its superb performance, FRP's growth in civil/structural markets has been kept somewhat in check by its high initial cost (as compared to conventional materials like steel and concrete). While this control on growth is a reality, the FRP industry is making significant strides in those applications where FRP's characteristics offer distinct advantages over conventional materials.

Research on FRP bridge decks began in the early 1990's as aerospace companies looked for alternative uses for their advanced products. By the mid-1990's, FRP gained acceptance from the bridge community as decks were applied on small, low volume demonstration projects. Since that time, FRP decks have been utilized on over ninety vehicular bridges in the United States (2). Designers have become more familiar with FRP's opportunities and have begun to apply it to those projects that best utilize its benefits – those cases where low weight, corrosion resistance, or rapid installation are critical. Specifically, FRP decks are great fits on historic bridges, movable bridges, and those with high traffic volumes.

Introduction to Broadway bridge

The Broadway Bridge over the Willamette River in Portland, OR is in the heart of the city's harbor and is a vital structure to its surrounding areas. This historic bridge carries four lanes of traffic with an average daily volume of 30,000 vehicles. With each of the bascule leaves measuring 140', the Broadway Bridge is also the seventh longest bascule bridge in the world.

Fig. 1: Vital to Portland's river and vehicular traffic, the Broadway Bridge's bascule span utilized a steel grid deck before being replaced by a solid-surface FRP deck in August 2004.

Built in 1912, the bridge has served Portland's marine (river) traffic and vehicular traffic quite well for over ninety years (3).

However, its age and frequent use left it with a long list of repair needs. One element that justified considerable attention was the bascule span's steel grid deck (4).

The years of aggressive traffic loading rendered the grid worn and less skid-resistant. When combined with the bridge's vertical curve, limited site distance, and traffic signal near its approach, the low skid-resistance caused significant safety concerns. Thus, the grid was selected for replacement.

The selection and execution of the grid replacement, however, was no easy task. In order to minimize costly, complicated re-work of the bascule span's mechanical drive system, designers needed to match the weight of the existing steel grid as closely as possible. While direct replacement with new grid was an option, the owner preferred an alternative solid surface deck in order to accommodate an improved skid-resistant wearing surface. Another significant rehabilitation challenge involved construction time: the new deck must be installed fast. River commuters and vehicular commuters alike required as much access to the bridge as could be allowed.

Because of the bridge's critical importance to commerce and the traveling public, its owner applied strict limitations to the project's construction schedule, holding the bridge's vehicular traffic closure to a defined 60-day period. During this 60-day period (in Summer 2004), the contractor executed key portions of the extensive structural rehabilitation - including steel repair, lead paint abatement, and the grid deck replacement. While the bridge was closed to vehicular traffic, the bridge was also available to river traffic on regular intervals. As such, the bascule span was available for opening every fourth day within the aggressive 60-day vehicular closure. This meant that a new deck must not only be installed quickly, but must be modular in nature and securable if a bascule opening were required during construction. Significant liquidated damages awaited the contractor if these requirements were not met.

With these requirements in mind, the bridge owner selected DuraSpan® FRP bridge deck system (manufactured by Martin Marietta Composites) to replace the worn steel grid on the bascule span. DuraSpan's low weight, pre-fabricated, solid-surface design allowed FRP to address the project's key parameters. Other DuraSpan properties offered ancillary benefits – including its resistance to corrosion, fatigue, and creep. Also, DuraSpan could be customized to accommodate certain key features of the existing bridge.

Details

Many of DuraSpan's details were designed to mimic those of conventional materials, particularly those of precast concrete panels. As such, the deck-to-beam connections look quite familiar to those with conventional deck design experience.

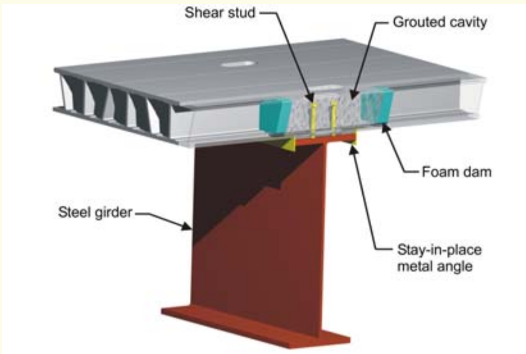

Fig 2: The new FRP deck was attached to the longitudinal steel girders via conventional methods.

Broadway Bridge utilized conventional shear studs and grout-filled cavities to connect the new deck to the bridge's longitudinal beams. Grout was poured through the deck into a cavity formed by stay-in-place metal angles, providing a variable haunch along each longitudinal beam. This attachment method had a proven track record in static testing, fatigue testing, and in-place performance. All work was performed from above.

DuraSpan panels arrived at the job site pre-fabricated into 8' x 46' modules, ready for installation on the beam's variable haunches. The length of the panel (46') matched the width of the bridge deck, as DuraSpan panels span perpendicular to the bridge's longitudinal beams. Shop workers pre-drilled all holes to accommodate the connections to the bridge's longitudinal beams. At the heel of each bascule leaf, the FRP deck interfaced with a concrete transition deck, which was designed to accommodate dynamic vehicular forces. At the bridge's center open joint (2"), the deck interfaced with heavy steel angles to handle dynamic forces. At its side edges, workers bonded an FRP curb to the deck along its full length.

Deck designers were able to incorporate a key feature of the existing bridge's geometry – its cross section. The pultruded panels were cambered (2 ¼") to match the parabolic crown on the bridge's approach spans. All cambering was handled in the shop and panels arrived at the job site in their "curved" state. Another key geometric feature of the existing bridge was its vertical alignment - the panels accommodated a vertical curve in the longitudinal stringers.

With no established AASHTO design criteria for FRP decks, the supplier took full responsibility for the design and performance of their propietary system. The supplier's criteria was developed using knowledge gleaned from traditional material guidelines (for concrete, steel, etc), FRP physical testing, and finite element analysis (FEA). Additionally, the supplier offer sealed contract drawings and shop drawings of their system as an added measure.

Installation

The project's construction sequence (when incorporating the extensive painting portion and other issues) forced the contractor to establish deck staging areas several hundred feet from the bascule leaves, but still within Broadway's truss. Two forklifts walked each panel from its staging area to its eventual resting place while maintaining extremely tight clearances on either side. These dual forklifts started at the heel of each span and worked towards the center of the bridge, walking over previously set panels. Each panel's low weight (typically under 6,000 lbs) allowed the contractor to utilize light duty, high performance equipment.

All of the bascule span's 32 deck panels (over 250 linear feet) and half of the shear studs were in place after two shifts. Miscellaneous tasks (including FRP splice strips at field joints and the remainder of the shear studs) were completed on the third day. Panels were correspondingly secured to the beams with temporary attachments and ready for a bascule opening on the fourth day.

On subsequent 3-day construction cycles, workers completed the installation by placing the grout, curb, and a thin polymer concrete wearing surface. In early September 2004, Multnomah County re-opened the Broadway Bridge to vehicular traffic while the extensive painting project continued into the near future.

Fig. 3: The Bridge's longitudinal stringers were prepared for placement of FRP panels.

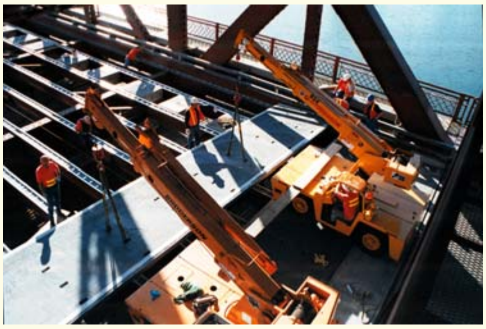

Fig. 4: FRP deck panels were set in place with dual forklifts.

Fig. 5: Pre-fabricated FRP deck panels were easy to handle and easy to install.

Fig. 6: All FRP panels were in place after two shifts.

Conclusion

While still a growing market segment, FRP decks are quite viable in certain circumstances. It is a singular solution to a complex set of demands that simply has not been available to owners and designers for a long period of time. The aggressive demands of the Broadway Bridge will prove to be an excellent case study as FRP decks are applied to more and more projects that harness their unique benefits.

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An Owner's Perspective

In each issue the newsletter will highlight a perspective from the "owner/user" community. In this issue John Hooks of the Federal Highway Administration and David Reynaud of the Civil Engineering Research Foundation discuss details of a test protocol developed under the aegis of the HITEC program. The protocol provides standards for testing and assessment of FRP bridge decks at both the structural and materials levels.

Standardized Protocol for the Assessment and Qualification of FRP Bridge Decks

by
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Since FRP decks are being developed by a number of companies and there is to-date no AASHTO code pertaining to these there was a level of concern related to their use even on demonstration projects. In order to provide a uniform and standardized methodology for the assessment of performance a comprehensive set of test protocols, design requirements, and assessment criteria were developed, through collaboration between State and Federal Transportation officials, and leading experts in bridge design and FRP use therein, under the aegis of the Civil Engineering Research Foundation (CERF) of the American Society of Civil Engineers (ASCE). In this article a brief summary of the methodology, developed under the resulting Highway Innovative Technology Evaluation Center (HITEC) protocol is provided.

In order to assess subcomponent performance and to ensure that mechanisms of failure are non-catastrophic tests to shear, flexural, and bearing response are required. The assessment of shear response is on a specimen of small span. The specimen is required at a minimum to support, without degradation, AASHTO factored loads. Flexural response is assessed through loading in a simply supported configuration over a span similar to that being considered for the field application. Loads at service level are to be introduced through current AASHTO tire patch configurations and again at a minimum no distress must be noted till the service load level with deflections not exceeding those of a conventional RC specimen of similar span. Bearing tests are required over local areas to ensure that no premature rib/web buckling would occur.

Since fatigue is a critical performance parameter for bridge decks, performance is required to be assessed at both the subcomponent and assemblage levels. At the assemblage levels full-scale assessment of performance is conducted using deck panels configured as shown in Figure 1 so as to have two joints between deck panels. The panels are simply supported over three girders to ensure assessment of continuity over the central support. Load, at a level of 1.5 times the design wheel load is to be introduced centrally between girders on the middle deck panel for 2 million cycles using a frequency not higher than 3 hertz (to avoid inadvertant damage from high cycle fatigue not expected in the field). In order to avoid local damage due to reversed load effects during cycling unloading should be stopped at 10% of the maximum load. The test is to be performed with a complete wearing surface and/or overlay using materials specified for field use. At the minimum strains and deflections are to be monitored every 25,000 cycles along with careful checking of joints for signs of distress. No distress (in the form of delamination, debonding or cracking) should be noted at the end of the test period and permanent deflection of the deck at midspan between girders should not exceed the initial set, prior to testing, by more than 10%. Since the integrity of joints is critical to stress transfer and for durability of the deck system it is expected that there would be no leakage of water through the joints, if placed to a nominal depth on top of the wearing surface after completion of the test.

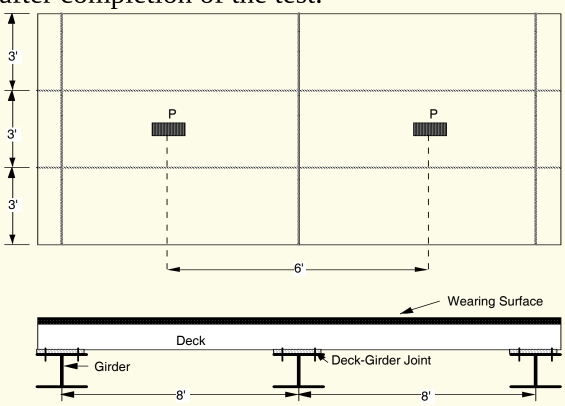

Figure 1: Schematic of Layout for Fatigue Test

In addition to the assemblage test, it is recommended that a 2-D finite element analysis, at a minimum, be conducted on the deck identity fatigue-critical subcomponents. Once these are identified these areas should be locally stressed in fatigue up to 5 million cycles at stress levels equivalent to those that would cause failure at the global level. As an example, these subcomponents are envisaged to include areas of internal bonded joints, sections with abrupt changes in geometry where strain levels under service loads are seen to exceed 20% of ultimate in the global 2-D model, and sections with cut-outs or mechanical connections for joints and connections. No fatigue induced crack propagation or delamination/separation of layers of fabric should be allowed in the subcomponents during the 5 million cycles of fatigue loading.

Stability and integrity of connections is critical to the efficient functioning of the deck system. For investigation of the deck-to-deck connections a single test is recommended using a configuration as shown in Figure 2 with loading on the two deck panels being alternated so as to induce differential movement across the deck-to-deck joint. A tie-down is used to induce continuity. The specimen is required to be cycled under factored loads of 1.3 [Dead Load +55/3 (Live Load + Impact Factor)], wherein impact loads are rated at 33%, for a minimum of 2 million cycles. It is important that each deck panel be of sufficient width to ensure slab action.

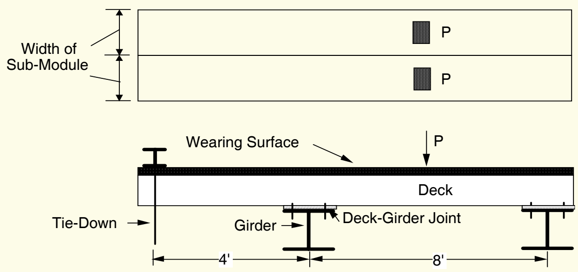

Fig 2: Schematic of Test Setup for Deck-to-Deck Connection

The deck-to-girder connection is to be tested through the use of a push-out test using a configuration similar to that shown in Figure 3. Load is to be introduced directly into the girder and strains monitored in the connections and decks. Care should be taken to ensure that decks do not crush locally under load at the bottom. Both monotonic loading, to failure load level, and fatigue cycling at service demand levels of the connectors shall be conducted. In addition to the push out tests, deck-to-girder connections should also be monitored during the deck-to-deck connection test described above, and results obtained should be compared. Monitoring should be through the use of appropriate means including strain gages, slip gages, and linear potentiometers and at the minimum shall assess aspects related to yielding of the connection (if metallic), strain in the connection assembly, and slippage and/or horizontal shear between the deck and the girder. The results of these tests should be analyzed through correlation of effects of stresses using a local-global FE model for the system. Strains are limited to a level of no greater than 20% of the ultimate strain under service loads i.e. Dead Load + Live Load + Impact Load, with maximum strains under dead load being limited to 10% of ultimate. In the case of occasional loading such as through application of permit loads, the maximum strain should be limited with a level of 50% of ultimate being deemed to cause overload at a level sufficient to risk premature failure of the system. In all cases where connectors are used it shall be shown that the connectors themselves meet all requirements of performance and that premature failure is not caused due to failure of the connector itself.

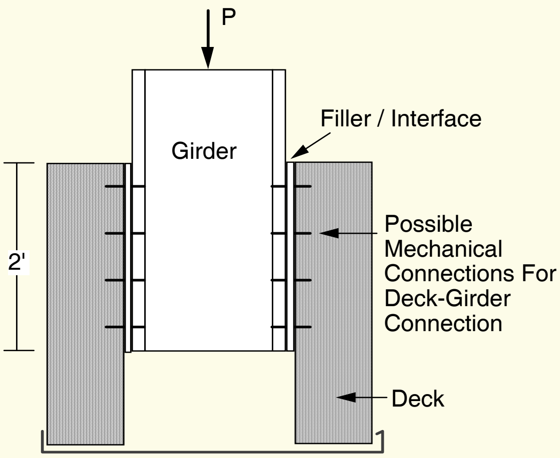

Fig. 3: Schematic of Test Set-up for Sample Deck-to-Girder Connection

The evaluation protocol includes a specified set of durability tests as well. The primary premise of this program is that the manufacturers will themselves provide a significant level of basic data pertaining to basic materials characteristics and durability. Tests conducted under this HITEC program will therefore concentrate on essential data emphasizing structural response and materials performance for structural behavior at predefined levels.

It is required that tests be conducted at two levels (a) coupons representative of the materials used in the deck panels and (b) subcomponents sized such that the specimen, when tested in bending, stresses the adhesive joint as well and hence evaluates the entire component consisting of adhesively bonded profiles (as appropriate).
Exposure environments to be considered in testing include

- (A) Ambient (as received)
- (W) Immersion in water at 78°F (ambient conditions)
- (M) Immersion in water at 110°F
- (H) Immersion in water at 140°F
- (D) Exposure to 140°F at a maximum of 50% RH
- (C) Immersion in cement extract (indicative of actual pH and chemical content)
- (FT) Exposure to Freeze/Thaw conditions with chlorides, and
- (S) Immersion in salt water at 78°F (ambient conditions)

Test values determined after exposure on average are required to meet 85% of the average as-received test values to be considered acceptable. Further, no single value used for the purposes of qualification shall be below 75% of the as-received value.

Coupons representative of the materials being used shall be characterized using tension, flexure and short-beam shear tests (following relevant ASTM procedures) using all environments listed above environments (A, W, M, H, D, C, FT, S). Glass transition temperatures will be determined through use of the Dynamic mechanical Thermal Analysis (DMTA) considering the peak of the tan d curve as the indicator.

Subcomponents of size sufficient to be representative of the actual deck shall be cut from a fulldeck fabricated using usual procedures and then shall be tested in three point bending under simply supported conditions after exposure to environments A, W, D, FT. As far as possible the subcomponents are to have holes and other attachments used for joints and connections. In the case of environment W the tests shall also be conducted after exposure in a "ponding" environment (i.e. water will be allowed to pond on top of the specimen, rather than the specimen being immersed in water). In addition to three point testing, samples shall be tested for fiber volume fraction (before exposure), moisture content and glass transition temperature. All tests should at minimum be conducted over a period of 36 months with specimens removed and tested at intervals of 3, 6, 12, 18, 24 and 36 months.

Further details on the protocol and test results from ongoing assessments can be found on the CERF/HITEC web site at http://www.cerf.org/hitec/eval/ongoing/decks.htm

Research Activities

In each newsletter ongoing research at a University, an Industrial Research Center, or a government laboratory, is highlighted. In this issue we highlight specific research activities at the Department of Civil Engineering, University of Patras, Greece

FRP Retrofitting of Rectangular RC Columns With or Without Corrosion

by

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Twenty concrete columns, with a 250-by-500 mm section and materials and detailing emulating older construction, have been tested at the University of Patras to investigate in a systematic way the effect of important parameters of seismic retrofit with FRP wraps, as well as the effect of rebar corrosion on the effectiveness of the retrofitting. As far as the number of FRP layers and the fiber material is concerned, it was concluded that replacing carbon fibers by glass fibers, while maintaining the same extensional stiffness of the FRP jacket in the circumferential direction, leads to about the same performance. Nonetheless, FRP extensional stiffness seems to be the controlling factor up to a certain limit, as increasing the number of CFRP layers from two to five does not materially improve performance. Previous damage left unrepaired reduced the effectiveness of rehabilitation with FRP wraps. Confinement by the FRP was very effective in increasing concrete strain capacity to levels of 5% to 6% even in the middle of a wide side of the column. Nonetheless, rectangular columns tested in the strong direction (with a 250 mm-wide compression zone) were found to benefit more from FRP wrapping than when tested in their weak direction (with a 500 mm-wide compression zone). Although wrapping with FRP is found to significantly improve seismic performance of columns which suffer from both lack of seismic detailing and of corrosion of the reinforcement, such corrosion materially reduces the effectiveness of FRP wraps as a strengthening measure, as the corroded bars become the weak link of the column, instead of the confined compression zone.

Fig. 1: Accelerated corrosion of RC columns.

Fig. 2: Experimental setup.

Fig. 3: Concrete crushing, and rebar buckling are combined with fracture of the FRP jacket at the base of the column.

Formation of IIFC Working Groups

A number of topic specific working groups have been formed within IIFC as detailed below. Those interested in participating are encouraged to contact the chairs of the working groups directly.

Bond between FRP and Concrete

Chairman: Prof. Jin-Guang Teng, The Hong Kong Polytechnic University
Email: cejgteng@polyu.edu.hk

Objectives

- To promote research collaboration on FRP-to-concrete bond behavior
- To provide a forum for exchange, discussion and consolidation of ideas and results through the organisation of specialist workshops and special sessions at conferences
- To establish design models
- To develop a state-of-the art report on the behaviour and modelling of bond between FRP and concrete within 3 years

Deliverables

- Proceedings of two international workshops in 2005 or 2006 and 2007 respectively
- State-of-the-art report presenting a review of existing knowledge and design recommendations in 2007
- Annual reports

FRP Bridge Decks

Chairman: Prof. Thomas Keller, Swiss Federal Institute of Technology Lausanne
Email: thomas.keller@epfl.ch

Objectives

- Development of a design guide for hybrid-FRP and all-FRP bridge superstructures, consisting of FRP bridge decks and steel, concrete or FRP main girders

Deliverables

- State-of-the-art report by December 2005
- Organization of two international workshops to get feedback on draft documents (2007/2008)
- Design guide – basis for design by December 2007
- Design guide – FRP specific chapters by December 2008
- Organization of an international conference to present the final design guide (2009)
- Annual reports

Shear Strength of FRP-Strengthened RC Members

Chairman: Dr. Jian-Fei Chen, The University of Edinburgh, UK
Email: : J.F.chen@ed.ac.uk

Objectives

- To promote research collaboration on shear strength of FRP strengthened concrete members
- To provide a forum for exchange, discussion and consolidation of ideas and results through the organisation of specialist workshops and special sessions at conferences
- To establish design models
- To develop a state-of-the art report on the behaviour and modelling of shear-strengthened concrete members within 3 years

Deliverables

- Proceedings of two international workshops in Years 2 and 3 respectively
- A state-of-the-art report by the end of Year 3
- Annual reports on the year's development
- Other specific reports

Hybrid Construction with Concrete

Chairman: Prof. Amir Mirmiran, Florida International University, USA
Email: mirmiran@fiu.edu

Objectives

- Develop understanding of cost-effective and innovative methods of combining FRP and concrete in various construction applications for bridges and buildings
- Maintain the state-of-the-art and the state-of-practice knowledge database on hybrid construction with FRP and concrete
- Develop necessary documentations, specifications, and guidelines that help engineers design and construct hybrid systems with FRP and concrete

Deliverables

- State-of-the-art report by March 2005
- Design guidelines by December 2006
- Construction specifications by December 2007
- Annual reports and updates

Ductility of FRP Plated Beams and Structures

Chairman: Prof. Deric Oehlers, University of Adelaide, Australia
Email: doehlers@civeng.adelaide.edu.au

Objectives

- To provide up to date information which can be used eventually to quantify the ability of an FRP plated structure to absorb energy and redistribute moment
- To promote research collaboration
- To eventually provide design rules

Deliverables

- A one off report on a review of the current state of knowledge up to the end of 2003
- Thereafter, annual reports on the year's development and updated recommendations for design if any

International Institute for FRP in Construction