AVOIDING DE-BONDING IN FRP STRENGTHENED REINFORCED CONCRETE BEAMS USING PRESTRESSING TECHNIQUES

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ABSTRACT

It has been widely accepted that carbon fiber reinforced polymers (CFRP\textsuperscript{s}) can be used effectively to strengthen reinforced concrete (RC) members. Until now most research and applications of such strengthening methods has been focused on gluing or bolting CFRPs to the members’ surface and the failure was often caused by premature de-bonding or de-lamination of the CFRP and therefore the superior tensile strength of CFRP could not be fully utilised. It is therefore logical to consider prestressing the CFRPs prior to its application for strengthening RC members. This paper presents the experimental results of flexural behaviour of five RC beams strengthened by prestressed-CFRP. It is found from the test results that there was no debonding of pretressed-CFRP occurred and the failure mode of the beams was the rupture of the CFRP and hence the CFRPs’ high tensile strength was fully utilized at the ultimate limit state. It is also found that the cracking load and flexural stiffness of the beams strengthened by prestressed-CFRPs were significantly increased. The reasons of why prestressed technique can be used to prevent debonding is explained which was supported by the test results. However there was no evidence that the prestressed-CFRP strengthening method will lead to higher ultimate load carrying capacity compared to non-prestressed-CFRP strengthening method. It is concluded that prestressed-CFRP can be effectively and economically used to strengthening RC beams to increase the cracking load, flexural stiffness and reduce deflection and to avoid debonding from occurring.

Keywords: de-bonding, prestressed, CFRP, strengthen; cracking load, flexural stiffness

INTRODUCTION

An increasing number of existing buildings and bridges overall the world are in need of strengthening or retrofitting due to factors such as deterioration, construction or design faults, additional load, or functional change (Zhang et. al. 2004). The strengthening or retrofitting has traditionally been accomplished using conventional materials and construction techniques, such as externally bonded steel plates, steel or concrete jackets and external post-tensioning (ACI440, 2002). Composite materials made of fibres in a polymeric resin, also known as fibre-reinforced-polymers (FRP) have emerged as an alternative to traditional materials and techniques and there are a number of externally bonded FRP systems commercially available in the market, such as Wet Layup FRP system, Prepreg FRP system, and Procured FRP system (ACI 440, 2002). The main advantages of the externally bonded FRP system include lightweight, noncorrosive and high-tensile strength of the FRP and these in turn provide a more flexible and economical technique than traditional steel-plate/jacket techniques, particularly in the areas with limited access.

Substantial experimental and theoretical research has been conducted over the last decade into the effectiveness of using FRP sheets/strips to strengthen or retrofit RC members and the behaviour of the strengthened structural elements, such as Grace et. al. (1999), Grace (2001), Teng et. al. (2002), ACI 440 (2002), Alagusundaramoorthy et. al. (2003), Tavakkolizdeh and Saadatmanesh (2003), El-Refaie et. al. (2003) and Ei-Hacha and Rizkalla (2004). Most of these investigation used non-prestressed method by gluing or bolting the FRPs to the members’ surfaces. However, the method of gluing or bolting FRPs to the members’ surface has a common problem of debonding or de-lamination and hence could not fully utilize the full tensile strength of the FRP, nor increase the members' flexural stiffness; and therefore not economical (Camata et. al. 2003, Lopez and Naaman 2003, Millar et. al. 2004 and Ye et. al. 2004). Millar et. al. (2004) claimed that “flexural strengthening of concrete structures with CFRP plate systems are the state of art nowadays… but numerous tests world-wide have shown that to a maximum, only 60% of the tensile strength of CFRP plates can be mobilized in ultimate limit state because the
bond strength which depends primarily on the tensile strength of the concrete cover is insufficient to develop the tensile strength of CFRP plates.” Similarly, Ye et al. (2004) stated that “it is known from a lot of research that the strengthen method using FRP can only be effectively developed after the yielding of tensile steel reinforcement in RC beams for flexural strengthening and it is natural to consider applying prestressed techniques for flexural strengthening”.

Based on the concepts and principles of prestressing techniques and the FRP bonding techniques, some researchers have attempted to prestress the CFRP sheet prior to its adhesion to the members’ substrate, such as El-Hacha et al. (2000 and), Wu et al. (2000), Wight and Erik (2001), Wu et al. (2002), El-Hacha et al. (2003), Wu et al. (2003) and Stoecklin and Meier (2003). The development of prestressed-CFRP technique was intended to capitalize on the high tensile capacity of the CFRP sheets and avoid de-bonding. This is because from a theoretical point of view, as advanced composite materials, CFRP has superiorly high tensile strength of more than 4000 MPa (which is much higher than that of steel) and their modulus of elasticity may be in the range of 150 to 230 GPa (which is about the same as that of steel). Such material properties possesses a requirement to CFRP that it must go through a large strain development in order to fully utilize the strength potential. However, when CFRP is applied externally (through adhesion using epoxy resin) to the RC members and to work together with the internal steel reinforcement and concrete, the CFRP may have only achieved 15% of its ultimate tensile strength when the steel yield and such limited utilization of its tensile strength could not contribute to the beams’ stiffness to reduce the structural deformation nor the development of cracks (width and pattern) (Triantafillou and Deskovic 1991). In addition the epoxy resin in the interface between the concrete and the CFRP for adhesion purpose has limited shear transfer ability (Wu, 2000) and the excessive use of epoxy resin in the interface will lead to reduction of the shear transfer ability and hence limited the utilization of the CFRP. According to Stoecklin and Meier (2003), the prestressed-CFRP method can relieve the stresses in the steel reinforcement and reduce crack width and deflection and it results a better fatigue behaviour of the structure.

By means of experimental tests, this research aims to investigate the flexural behavior of the RC beams strengthened by prestressed-CFRP with the hope to overcome the above mentioned shortcomings of the conventional FRP-strengthening method.

EXPERIMENTAL PROGRAM

Five beams, four with prestressed-CFRP sheet and one non-prestressed-CFRP were constructed and tested. All beams have the same dimension of 100 x 150 x 2200 mm (width x depth x length) as shown in Figure 1. The span of the beams was 2000 mm. The steel reinforcement ratio was 0.67%. The area of CFRP sheet used was 90 mm wide with an average thickness of 0.167 mm. One layer of CFRP was applied to the RC beams. The properties of the concrete and steel obtained from tests are summarized in Table 1.

<table>
<thead>
<tr>
<th>Material</th>
<th>Strength (MPa)</th>
<th>Modulus of elasticity (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP sheet tensile strength</td>
<td>2941</td>
<td>207200</td>
</tr>
<tr>
<td>Longitudinal steel bars (8 mm diameter) tensile strength</td>
<td>309.9</td>
<td>224500</td>
</tr>
<tr>
<td>Steel stirrups (cold-drawn steel) yield strength</td>
<td>595</td>
<td>201200</td>
</tr>
<tr>
<td>Concrete cube compressive strength</td>
<td>24.9</td>
<td>29700</td>
</tr>
</tbody>
</table>

The test variables include the effect of the different level of prestress induced by prestressing the CFRP-sheet on the member’s flexural stiffness, cracking load, deflection, ultimate load carrying capacity and ductility of the strengthened member (refer to Table 2 for details). The loading arrangement is shown Figure 1. Four point loading test was used and the loads were applied to the beams from the bottom upwards and therefore the prestressed-CFRP sheet was placed on the top side of the beam.

Strain gauges were used during the prestressing process to measure the actual tensile force in the CFRP sheets. The details of this prestressing set up is described separately and can be found in Shang et al (2005) while an overall picture of the anchorages and stressing equipment is shown in Figure 2.

The beams were tested after 10 days of prestressing the CFRP sheets. During testing, five dial gauges were used to measure deflections at the mid-span, at loading points and at the supports. The beams’ curvatures were then derived from these five deflection measurements. Strain gauges were used to measure the strains of the concrete, the CFRP sheets and the internal steel reinforcement (note the strain gauges were installed to the internal steel reinforcement at the time of casting the beams).
Table 2 Beam details

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Prestress (N)</th>
<th>CFRP initial stress due to prestressing (MPa)</th>
<th>Prestressing level (% of FRP’s ultimate strength)</th>
<th>CFRP initial strain due to prestressing (10^-6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>B2</td>
<td>9000</td>
<td>588</td>
<td>20</td>
<td>3300</td>
</tr>
<tr>
<td>B3</td>
<td>16000</td>
<td>1046</td>
<td>35</td>
<td>6000</td>
</tr>
<tr>
<td>B4</td>
<td>16000</td>
<td>1046</td>
<td>35</td>
<td>6000</td>
</tr>
<tr>
<td>B5</td>
<td>22000</td>
<td>1438</td>
<td>50</td>
<td>8400</td>
</tr>
</tbody>
</table>

TEST RESULTS AND DISCUSSIONS

Cracking Load, Yield Load and Ultimate Load Carrying Capacity

Table 3 provides a summary of the test results of all beams. It is clear that the cracking loads increased significantly for the RC beams strengthened by prestressed-CFRP sheets (ie beams B2 to B5). For example, if compared Beam B3 to Beam B1, the cracking load increased twice from 4 kN to 9 kN, when a prestress of 16 kN was applied and there was an increase of 275% when a prestress of 22 kN was applied (refer to Beam B5 to B1). Similar increase to the yield loads (ie the load level at the time that the steel reinforcement yielded), there was significant increase of yield load (up to 1.7 times) with the application of prestress. Furthermore, the ultimate strains of the CFRP sheets were in the range of 1.2% to 1.6% which was close to its ultimate strain and...
this shows the effective utilization of the CFRP material and its strength. However there is no obvious evidence that the ultimate load carrying capacity will increase with the application of prestress.

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Prestress (kN)</th>
<th>Cracking load (kN)</th>
<th>R1 (%)</th>
<th>Load at yield of steel (kN)</th>
<th>R2 (%)</th>
<th>Ultimate load carrying capacity (kN)</th>
<th>R3 (%)</th>
<th>Ultimate mid-span deflection (mm)</th>
<th>CFRP strain at ultimate limit state (10^6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>0</td>
<td>4</td>
<td>100</td>
<td>12.0</td>
<td>100</td>
<td>22.5</td>
<td>100</td>
<td>28.71</td>
<td>11931</td>
</tr>
<tr>
<td>B2</td>
<td>9000</td>
<td>5</td>
<td>125</td>
<td>14.0</td>
<td>117</td>
<td>21.5</td>
<td>96</td>
<td>24.28</td>
<td>12501</td>
</tr>
<tr>
<td>B3</td>
<td>16000</td>
<td>9</td>
<td>225</td>
<td>17.0</td>
<td>142</td>
<td>28.0</td>
<td>124</td>
<td>28.84</td>
<td>15556</td>
</tr>
<tr>
<td>B4</td>
<td>16000</td>
<td>8</td>
<td>200</td>
<td>20.0</td>
<td>167</td>
<td>26.0</td>
<td>116</td>
<td>23.11</td>
<td>13023</td>
</tr>
<tr>
<td>B5</td>
<td>22000</td>
<td>11</td>
<td>275</td>
<td>20.0</td>
<td>167</td>
<td>25.0</td>
<td>111</td>
<td>16.27</td>
<td>15950</td>
</tr>
</tbody>
</table>

Note: R1, R2 and R3 are ratios of the cracking load, yielding load, and ultimate load between prestressed beams to non-prestressed beams respectively.

**Load-Deflection Relationships**

Figure 3 shows the load-deflection relationships of the tested beams. It is clear that with the increase of prestress, there were significant increases of cracking loads. This means that the application of prestress has significantly increased the beams’ stiffness at working load condition and hence the advantage of applying the prestress is clear. Furthermore, the figure also shows that in general, with the increase of the level of prestress, there is an increase of beams’ stiffness and hence a decrease of deflection (at the same loading level).

**Failure Modes and Energy Release**

Two failure modes were observed in this study: rupture of CFRP sheet and crushing of concrete. The failure of Beams B2, B3, B4 and B5 were due to the rupture of the prestressed-CFRP sheet while the Beam B1 was due to crushing of concrete (then followed by rupture of CFRP sheets). These failure modes were also confirmed by the measured CFRP strains at the ultimate limit state (Refer Table 3). For the failures that were caused by the rupture of the CFRP sheet, at the time of the rupture of the CFRP sheet, substantial strain energy was released and accompanying a loud rupture noise/sound. The released energy caused the concrete to spall off at the mid-span region and the steel reinforcement was exposed. At this moment, there was a rapid increase of deflection while there was a drop in load carrying capacity. Figure 4 shows the actual failure modes: Figure 4(a) shows the failure due to crushing of concrete at the soffit of the beam (note the load was applied from bottom of the beam.
upwards). It also shows that the CFRP sheet ruptured and the steel reinforcements are exposed due to the spalling of concrete. The cracking pattern was also shown in the figure where many small cracks were formed around on the tension side of the beams with a few major cracks developed near the ultimate limited state. In Figure 4(b) another failure mode was shown which was due to rupture of the CFRP sheet but accompanied with the concrete cover debonding (a major horizontal crack) at the level of the internal steel reinforcement. This phenomenon indicates the epoxy resin (glue) was sufficient but the concrete cover was insufficient. Such failure mode requires further research to determine a minimum coverage for the steel reinforcement in order to avoid such failure.

(a) Failure due to crush of concrete and rupture of CFRP (note the load was applied from bottom of the beam and hence the top side of the beam was under tension and the soffit under compression)  
(b) Failure due to rupture of CFPR sheet and accompanied by spalling of concrete/horizontal crack in the level of the concrete and internal steel reinforcement.

Figure 4 Different failure modes observed from the experimental tests

**Effect of Prestressing on Deflection and Flexural Stiffness**

The test results show that the yield load of the reference beam (B1) was about 17 kN. In order to investigate the effect of level of prestressing force on the members flexural stiffness, a summary of the beam behaviour has been produced (Table 4) using the 17 kN as the referencing loading. It can be seen from Table 4 that the application of prestress has provided a major effect to the members' deflection and flexural stiffness. For example, the deflection dropped from 15.65 mm (at load level of 17 kN) to 7.63 mm which was almost a 50% reduction, when a prestress of 16 kN was applied to the beam. This was also shown in Figure 4 of the load-deflection curves. The reduction of deflection was mainly achieved through the increase of stiffness via the increase of the cracking load and yield load as the result of the application of prestress. This is verification that the application of prestressed-CFRP sheet become a viable method to significantly improve the members’ serviceability (ie the reduction of deflection under serviceability limit state).

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Deflection at load level of 17 kN (mm)</th>
<th>R4 (%)</th>
<th>CFRP ultimate strain $\varepsilon_{u}$ ($10^{-6}$)</th>
<th>R5 (%)</th>
<th>CFRP strain at yield load $\varepsilon_{y}$ ($10^{-6}$)</th>
<th>R6 (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>15.65</td>
<td>100.0</td>
<td>11931</td>
<td>100.0</td>
<td>1863</td>
<td>13.1</td>
</tr>
<tr>
<td>B2</td>
<td>9.26</td>
<td>59.2</td>
<td>12501</td>
<td>104.8</td>
<td>5387</td>
<td>37.9</td>
</tr>
<tr>
<td>B3</td>
<td>7.32</td>
<td>46.8</td>
<td>15556</td>
<td>130.4</td>
<td>8100</td>
<td>57.0</td>
</tr>
<tr>
<td>B4</td>
<td>7.60</td>
<td>48.6</td>
<td>13023</td>
<td>109.2</td>
<td>8800</td>
<td>62.0</td>
</tr>
<tr>
<td>B5</td>
<td>4.75</td>
<td>30.4</td>
<td>15950</td>
<td>133.7</td>
<td>10759</td>
<td>75.8</td>
</tr>
</tbody>
</table>

Note: R4, R5 and R6 are the ratios of the deflections, ultimate strains and strains at yield load of the prestressed beams (B2, B3, B4, and B5) to the reference beam B1.

**CFRP’s Strain at Ultimate Limit State**

At ultimate limit state, the strain of the prestressed-CFRP has two components: the initial strain induced by the prestress and the strain induced by the applied load (note the creep strain of the CFRP was very small or
practically zero therefore can be ignored). It is clear from Table 4 that the CFRP’s ultimate strains of all prestressed-CFRP were larger than the reference beam B1 (apart from B5 due to experimental defects). This is due to the initial prestress and the fact that there was no debonding occurred in the prestressed-beams in the entire loading process. The high strains of up to 1.59% of the prestressed-CFRP at ultimate limit state is a good evidence of the better utilisation of the CFRP material which is an advantage of this method.

Why no De-Bonding in Prestressed-CFRP Strengthened RC Beams

During the tests, it was observed that the failure of the reference beam (B1) was due to the debonding in the gluing interface between the CFRP sheet and the concrete surface while Beams B2 to B5 did not have such debonding. The application of prestressed-CFRP sheet has effectively avoided the debonding in the interface between CFRP and concrete. There are two reasons for this: (1) prestressed-CFRP sheets in Beams B2 to B5 undergo less stress-strain development at the ultimate limit state (compared to the non-prestressed-CFRP sheets) and hence has less chance for debonding and (2) in the reference beam B1, the non-prestressed-CFRP sheet is well adhered to the concrete surface using the epoxy resin, in such situation, as shown in Figure 5, when a curvature is induced to the beam by the applied load, the CFRP sheet will be subjected to a debonding force $N$ which is perpendicular to the curvature (as shown in the Figure 5), the force $N$ is obviously a contributor to the debonding of the CFRP from the concrete surface. However, for the prestressed-CFRP, since the CFRP was stressed/stretched and therefore will not have the same curvature of the beam and therefore there will be no such a debonding force $N$.

Figure 5 The debonding force $N$ in the non-prestressed-CFRP sheet due to members’ curvature under load

CONCLUSIONS

Based on the results obtained from the experimental tests, the following conclusions can be reached. (1) Compared to the non-prestressed-CFRP method, the prestressed-CFRP method can significantly increase the cracking load and flexural stiffness and therefore reduce deflection at working load level; (2) under the serviceability limit state (or under working stress limit state in bridge design), this method can also allow the better use of the CFRP’s high tensile strength; (3) at the ultimate limit state, this method can effectively prevent the debonding failure (which was often a failure mode of the traditional FRP strengthening method); (4) A failure mode of rupture of the prestressed-CFRP sheet (rather than de-bonding) is achievable and hence lead to a full utilization of the CFRP’s high tensile strength. In short, the use of prestressed-CFRP sheet is a feasible and effective method to strengthen concrete structures.

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