SHEAR BEHAVIOR OF PRESTRESSED DECKED BULB T BEAMS REINFORCED WITH CFCC STIRRUPS

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ABSTRACT

Carbon fiber composite cable (CFCC) reinforcement has been broadly accepted as a non-corrosive alternative to the conventional steel reinforcement in bridge construction. Although the flexural behavior of beams prestressed with CFCC strands has been thoroughly examined, their shear behavior when steel stirrups are replaced with CFCC stirrups remains uncertain. This study outlines an extensive research program conducted to evaluate the shear behavior of prestressed decked bulb T beams with CFCC stirrups. The investigation addresses the effect of the shear-span-to-depth (a/d) ratio and the type of transverse reinforcement on the shear carrying capacity. Four 9.45-m-long beams reinforced and prestressed with CFCC reinforcement were constructed and tested under shear loading to failure. Half of the span of each test beam was reinforced with CFCC stirrups and the other half was reinforced with steel stirrups. To assess the performance of CFCC stirrups versus that of steel stirrups, both ends of each beam were tested under identical shear load. The obtained experimental results were compared with developed finite element (FE) models to validate the results. Beam ends with CFCC stirrups failed due to either concrete web crushing or top concrete compression failure, while beam ends with steel stirrups failed in shear tension mode due to yielding of stirrups. The developed FE models predicted the shear behavior and capacity of the test beams with a difference of less than 10%. This investigation demonstrates that CFCC stirrups can be regarded as a non-corrosive alternative to steel stirrups in highway bridge girders.

KEYWORDS

Carbon Fiber Composite Cable, Shear, Prestressed, Decked T beams.

INTRODUCTION

Accelerated corrosion rates are more pronounced in prestressed concrete (PC) structures than in reinforced concrete (RC) structures due to high stress levels in the prestressing strands (Vu et al. 2009). In order to address corrosion related problems in PC structures, non-corrosive reinforcement materials such as carbon fiber composite cable (CFCC) has emerged as an innovative and efficient alternative to conventional steel reinforcement. Extensive research work has been conducted to evaluate the flexural behavior of beams prestressed/reinforced with longitudinal CFCC strands (Grace et al. 2012, 2013). However, the use of CFCC as shear reinforcement in the form of stirrups in prestressed beams has not been thoroughly examined. Shear behavior of concrete beams with CFCC reinforcement is remarkably different from that of beams with steel reinforcement. Joint ASCE-ACI committee 445 for shear and torsion (ASCE-ACI 1998) identifies most important parameters influencing shear capacity as: (a) depth of member or size effect, (b) shear-span-to-depth ratio and supporting conditions, (c) longitudinal reinforcement type, (d) axial force or level of prestressing, (e) compressive strength of concrete, and (f) stirrup spacing. Various shear design equations based on different empirical and theoretical approaches have been developed to address some or all of these factors and to evaluate the shear performance of beams with CFCC stirrups. However, available data are insufficient and/or inadequate to merge the influence of all these factors into a single unified shear design guideline for CFCC prestressed and reinforced concrete members.

The current study aims at evaluating the shear performance of beams with CFCC stirrups with different shear-span-to-depth (a/d) ratios. According to ACI-318 (ACI Committee-318 2011), the shear strength of concrete
members is significantly influenced by a/d ratio. This effect is recognized in shear design equations for beams with steel stirrups by including the shear-moment interaction while calculating the concrete shear resistance. Some previous studies (Razaqpur et al. 2011; El-Sayed et al. 2006) have reported the effect of a/d on the shear resistance of FRP reinforced concrete members. However, these studies addressed FRP reinforced concrete members without transverse reinforcement.

**EXPERIMENTAL PROGRAM**

Four decked bulb T beams were constructed, instrumented, and tested under shear loading with a/d ratios of 3, 4, 5, and 6. Half of the span of each beam was provided with CFCC stirrups, while the other half was provided with steel stirrups. Both ends of each beam were tested under the identical shear loading setup to compare the performance of CFCC stirrups to that of steel stirrups. The beams were identical in dimensions and cross section (shown in Figure 1) with a top flange width of 457 mm, a bottom flange width of 305 mm, an overall depth of 406 mm, a web thickness of 76 mm and an effective span of 9,450 mm. Each beam was pre-tensioned with four CFCC strands. Each strand had a diameter of 15.2 mm, an effective cross sectional area of 116 mm², and was pre-tensioned with an initial effective prestressing force of 111 kN. The CFCC stirrups had a diameter of 10.5 mm, while the steel stirrups had a diameter of 10 mm (M10). All the stirrups were provided at a center-to-center spacing of 150 mm. The material properties of the reinforcements are shown in Table 1.

![Figure 1. Cross-section of prestressed decked bulb T beam](image)

<table>
<thead>
<tr>
<th>Material Property</th>
<th>CFCC Longitudinal Reinforcement</th>
<th>CFCC Stirrups</th>
<th>Steel Stirrups</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter (mm)</td>
<td>15.2</td>
<td>10.5</td>
<td>10.0</td>
</tr>
<tr>
<td>Effective Cross Sectional Area (mm²)</td>
<td>116.0</td>
<td>58.0</td>
<td>71.0</td>
</tr>
<tr>
<td>Yield Strength (MPa)</td>
<td>--------</td>
<td>--------</td>
<td>413</td>
</tr>
<tr>
<td>Tensile Strength (MPa)</td>
<td>2,930</td>
<td>2,840</td>
<td>621</td>
</tr>
<tr>
<td>Elastic Modulus (MPa)</td>
<td>148,996</td>
<td>150,030</td>
<td>200,000</td>
</tr>
</tbody>
</table>

The prestressing forces were applied through a prestressing bed consisted of two steel bulkheads rigidly anchored to a heavily reinforced deep concrete foundation. Newly developed mechanical anchorage system (Grace et al. 2012) was used at each end of the CFCC strands to reduce the prestressing loss and strand damage. Figure 2(a) shows the CFCC reinforcement cage in the formwork. The concrete mix was designed to achieve an average 28-day compressive strength of 62 MPa. The beams were allowed to cure for 7 days using wet burlaps and plastic sheets to maintain moisture. The pre-tensioning force was continuously monitored from time of casting until release after 7 days. The concrete compressive strength averaged around 44 MPa at the time of
prestress release. The release of the prestressing force was executed by slowly heating the steel strands, attached to the CFCC strands through special couplers, with acetylene torch.

![Figure 2. CFCC prestressed decked bulb T beam construction and test set up](image)

**Instrumentation and Test Setup**

Strain gauges were attached to both non-prestressing and prestressing CFCC strands near the loading point to measure the strains during prestressing and load testing of the beams. Strain gauges were also attached to the vertical leg of the stirrups and to the concrete surface at the top flange and at the mid depth of the web. Linear motion transducers (LMTs) were attached at the soffit of the beam under the loading point to capture the deflection during test. Linear variable differential transducers (LVDTs) were mounted on the web within the critical shear span in sets of three LVDTs arranged at $0^\circ$, $45^\circ$, and $90^\circ$ directions. The recorded readings were later used to determine shear crack width using an equation given by Shehata (1999). All of the sensors were calibrated and connected to a central digital acquisition system using a software interface.

The beams were simply supported over two reinforced neoprene bearing pads and were subjected to a concentrated vertical load, applied by an MTS 1000-kN-capacity hydraulic actuator. The shear span was measured from the center of the support to the loading point and was adjusted to 1140, 1520, 1910, and 2160 mm, which corresponded to a/d ratios of 3, 4, 5, and 6, respectively. Figure 2(b) shows instrumentation and shear test setup of a prestressed decked bulb T beam.

The shear-testing scenario included testing the beam end with CFCC stirrups to failure, followed by testing the beam end with steel stirrups to failure. During the test, the load was applied through several loading and unloading cycles with an increment of 9 kN per load cycle. Crack mapping was performed at the end of each loading cycle using a Comparometer® to record the initiation and propagation of cracks. The last loading cycle included loading the beam end to failure.

**TEST RESULTS AND DISCUSSIONS**

Table 2 summarizes the deflection, the strain in stirrups and concrete, and the failure modes of the test beams. The letter in the beam acronym refers to the material of stirrups whether it is CFCC (C) or steel (S), while the number denotes the a/d ratio. Beam ends with a/d ratios of 3 and 4 initially exhibited shear cracks at the mid-depth of the web within the critical shear span, whereas beam ends with a/d ratios of 5 and 6 exhibited flexural cracks at the soffit of the beam under the loading point. With increasing the applied load, new shear and flexural cracks formed within the critical shear span while existing cracks widened and propagated towards the loading point. The failure mode was influenced by both the type of stirrups and the a/d ratio. Beam ends reinforced with CFCC stirrups and with a/d ratio of 3 or 4 failed due to loss of diagonal concrete compression strut, which resulted in web crushing failure. Whereas, beam ends reinforced with CFCC stirrups and with a/d ratio of 5 or 6 failed due to flexural concrete crushing in a shear compression failure mode. On the contrary, beam ends reinforced with steel stirrups and with a/d ratio of 3 or 4 failed by yielding of stirrups followed by web crushing, while beam ends with a/d ratio of 5 or 6 failed by yielding of stirrups followed by crushing of the concrete at the top flange. The difference in mode of failure can be related to the difference between CFCC and steel stirrups in the stress-strain relationship. With higher shear force, the steel stirrups exhibited a yield plateau represented as a
significant increase in strain accompanied by an insignificant increase in shear force. This resulted in widening the shear cracks and consequently the failure of the beam. On the other hand, CFCC stirrups did not exhibit any yield plateau as the strain in stirrups increased linearly with the increase in shear force. Ultimately, the shear cracks propagated and widened excessively and the stress in concrete reached the crushing strain.

The relationship between the applied shear force and the deflection of the test beams is shown in Figure 3(a). The relationship exhibited a bilinear behavior distinguished by the formation of cracks, which marked the loss in stiffness and consequently the change in slope. The material property of stirrups (CFCC/steel) did not seem to influence the shear force-deflection response of the beams.

Prior to concrete cracking, the shear force was entirely taken by the concrete section and the strain in the stirrups was negligible. At a/d ratios of 3, 4, 5, and 6, the cracking shear forces were 119, 85, 69, and 63 kN for beam ends with CFCC stirrups, and were 122, 89, 70, and 55 kN for beam ends with steel stirrups. It can be observed that the cracking shear forces of beam ends with CFCC stirrups were similar to those of beam ends with steel stirrups. In addition, the cracking shear force decreased with increasing the a/d ratio. The onset of the cracks led to a redistribution of stresses as the applied shear force, after cracking, was resisted by: the un-cracked portion of the concrete section, the stirrups, and the aggregate interlock through the crack.

The strain in stirrups of the test beams was plotted versus the shear force as shown in Figure 3(b). The number of stirrups involved in shear resistance varied with the change in a/d ratio. However, there was no rupture of the CFCC stirrups in any of the beams. Steel stirrups yielded before the failure of the beam ends. CFCC stirrups achieved a maximum strain of 3,588, 3,575, 3,467, and 3,426 με, while steel stirrups achieved a maximum strain of 3,730, 3,349, 3,652, and 3,782 με for a/d ratios of 3, 4, 5, and 6, respectively. The shear force at the yield of steel stirrups decreased with increasing a/d ratio. In addition, the maximum strain in the CFCC stirrups was significantly higher than the current 2,000 με design strain limit specified by ACI 440.4R-04 shear design guidelines for PC structures and it was less than the 4,000 με design strain limit provided by ACI 440.1R-06 for RC structures.

The maximum concrete strain at ultimate shear force ranged from -1,282 to -2,732 με for beam ends with CFCC stirrups, while it ranged from -1,642 to -2,629 με for ends with steel stirrups. The response of shear force vs. concrete strain also demonstrated a bilinear relationship with an apparent decrease in slope near concrete cracking shear force.

LVDTs were used to measure the crack width using a relationship given by Shehata (1999). Beam ends with CFCC stirrups exhibited maximum crack widths of 1.14, 1.09, 0.96, and 1.08 mm, whereas beam ends with steel stirrups exhibited maximum crack widths of 1.19, 0.86, 0.81 and 0.81 mm at a/d ratios of 3, 4, 5 and 6, respectively. The measured crack width at any particular a/d ratio was higher in case of beam ends with CFCC stirrups as compared to ends with steel stirrups. This can be attributed to lower stiffness of CFCC compared to that of steel. The number of cracks within any shear span increased with increasing the shear span.

### Table 2. Summary of the test results

<table>
<thead>
<tr>
<th>Beam</th>
<th>Cracking Shear Force (kN)</th>
<th>Ultimate Shear Force (kN)</th>
<th>Deflection (mm)</th>
<th>Maximum Concrete Strain at the Top (με)</th>
<th>Maximum Strain in Stirrups (με)</th>
<th>Mode of Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-3</td>
<td>Exp. 119</td>
<td>Exp. 136</td>
<td>Exp. 261</td>
<td>Exp. 264</td>
<td>Exp. 41</td>
<td>Exp. 43</td>
</tr>
<tr>
<td>S-3</td>
<td>Exp. 122</td>
<td>Exp. 136</td>
<td>Exp. 272</td>
<td>Exp. 282</td>
<td>Exp. 36</td>
<td>Exp. 49</td>
</tr>
<tr>
<td>C-4</td>
<td>Exp. 85</td>
<td>Exp. 100</td>
<td>Exp. 232</td>
<td>Exp. 236</td>
<td>Exp. 76</td>
<td>Exp. 78</td>
</tr>
<tr>
<td>S-4</td>
<td>Exp. 89</td>
<td>Exp. 98</td>
<td>Exp. 238</td>
<td>Exp. 249</td>
<td>Exp. 66</td>
<td>Exp. 71</td>
</tr>
<tr>
<td>C-5</td>
<td>Exp. 69</td>
<td>Exp. 82</td>
<td>Exp. 218</td>
<td>Exp. 221</td>
<td>Exp. 104</td>
<td>Exp. 104</td>
</tr>
<tr>
<td>S-5</td>
<td>Exp. 70</td>
<td>Exp. 81</td>
<td>Exp. 221</td>
<td>Exp. 227</td>
<td>Exp. 90</td>
<td>Exp. 104</td>
</tr>
<tr>
<td>C-6</td>
<td>Exp. 63</td>
<td>Exp. 74</td>
<td>Exp. 206</td>
<td>Exp. 218</td>
<td>Exp. 139</td>
<td>Exp. 139</td>
</tr>
<tr>
<td>S-6</td>
<td>Exp. 55</td>
<td>Exp. 74</td>
<td>Exp. 197</td>
<td>Exp. 208</td>
<td>Exp. 122</td>
<td>Exp. 130</td>
</tr>
</tbody>
</table>

WC = Web crushing failure, Y-WC = Yielding of stirrups followed by web crushing, SC-T = Shear compression failure at the top flange, Y-SC-T = Yielding of stirrups followed by shear compression failure at the top flange.
NUMERICAL ANALYSIS

Parallel to the experimental study, a finite element analysis (FEA) was performed using commercially available software ABAQUS. The beams were modeled using a three-dimensional eight-node linear brick element C3D8R. A continuum plasticity based concrete damage model was used to model the material property of concrete. The reinforcement, longitudinal and transverse, was modeled using a two-node linear 3D truss element (T3D2) with three degrees of freedom at each node. Fine details of finite element models can be found elsewhere (Grace et al. 2012). The numerical values for ultimate shear force, maximum deflection, maximum concrete compressive strain, and maximum stirrup strain were compared to those obtained experimentally and presented in Table 2. The deflection and strain response of the numerical beam models were in close agreement with the experimental results. In addition, the failure pattern in the FEA models agreed well with the experimental failure mode. An example of experimental versus numerical failure pattern is shown in Figure 4. Overall, it can be concluded that the FEA accurately simulated the shear performance of the test beams.

CONCLUSIONS

Based on the experimental and analytical results of the test beams, the following conclusions are drawn:

1. The mode of shear failure for beam ends with CFCC stirrups was influenced by the a/d ratio. For a/d ratios of 3 and 4, the failure was characterized by concrete crushing in the web, whereas it was crushing of the concrete at the top flange for a/d ratios of 5 and 6. Beam ends with steel stirrups failed in a shear tension mode due to yielding of stirrups followed by concrete crushing in the web, for a/d ratios of 3 and 4, or in the top flange, for a/d ratios of 5 and 6. Therefore, the a/d ratio or the shear-moment interaction is an important parameter in determining the concrete shear resistance of CFCC prestressed concrete members. The current ACI 440.4R-04 shear design guidelines need to be updated to include this parameter.
2. The ultimate shear carrying capacities of beam ends with CFCC stirrups were very close to those of beam ends with steel stirrups, although the mode of failure and the level of strain in stirrups were quite different. The failure shear force for beam ends with CFCC stirrups were 96, 97, 99, and 105% of the failure shear force of beam ends with steel stirrups for a/d ratios of 3, 4, 5, and 6, respectively.

3. The maximum strain in the CFCC stirrups at the ultimate shear force was much higher than the current 2,000 με design strain limit specified by ACI 440.4R-04 shear design guidelines but it less than the strain limit of 4,000 με defined by ACI 440.1R-06 shear design guidelines.

4. The low elastic modulus of the CFCC transverse reinforcement affected the strain distribution around the shear cracks and resulted in wider cracks than those observed in beam ends with steel stirrups.

5. The numerical models of the beams accurately predicted the overall shear performance of the test beams with an average difference of less than 10%.

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