Shear Strengthening of a PC Bridge Girder with NSM CFRP Rectangular Bars

Antonio Nanni*,1, Marco Di Ludovico2 and Renato Parretti3

1Jones Professor of Civil Engineering, University of Missouri – Rolla, 224 Engineering Research Lab, 1870 Miner Circle, Rolla, MO 65409-0710, Office 573-341-4497; fax -6215; E-Mail: nanni@umr.edu
2University of Naples Federico II Naples, Italy, Dipartimento Analisi e Progettazione Strutturale, E-mail: marcodiludovico@libero.it
3Co-Force America, Inc., 800 West 14 Street, Rolla, MO 65401, E-mail: parretti@coforceinternational.com

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Abstract: Bridges worldwide are falling into a state of disrepair caused by deterioration, traffic overloading, and poor maintenance. It is estimated that over 100,000 of the half million existing bridges in the US are in need of repair or replacement. Although the full replacement option could be viable, it is an expensive solution with significant disruption of service. This paper focuses on the use of carbon fiber reinforced polymer (CFRP) rectangular bars installed as near surface mounted (NSM) reinforcement for shear strengthening in conjunction with an externally bonded pre-cured CFRP laminate to increase the flexural capacity of a prestressed concrete (PC) bridge girder. The specimen, which was removed from an overloaded bridge in Kansas was strengthened and tested in the laboratory. Test results showed that the proposed NSM reinforcement technique represents an effective solution to increase shear capacity.

Key words: bridges, fiber reinforced polymer, flexural strengthening, prestressed concrete, structural repair, shear strengthening.

1. BACKGROUND

1.1. Specimen Characteristics

Numerous bridges throughout the state of Kansas utilize prestressed concrete (PC) members. In many cases, frequent overloading has occurred due to the heavier vehicles now traveling on these structures. This has led to significant cracking of the PC members and in some cases spalling of concrete (Peterman and Reed 2002). Because cracked PC members are susceptible to steel strand fatigue as well as corrosion, the damaged girders on these bridges need to be repaired or replaced.

One such bridge in which multiple overloads have occurred is Bridge #56 in Graham County, Kansas. The four-span bridge was composed of 1.8 m (6 ft) wide PC double-T members. The inspection of the bridge showed that most of the stems of the 12.2 m (40 ft) interior double-T girders were severely cracked and in some cases spalling had occurred. Because of the numerous existing bridges in need of upgrade and considering cost and inconvenience of replacing damaged bridge members, the state Departments of Transportation of Kansas and Missouri decided to support research in order to investigate the feasibility of repairing PC members with carbon fiber reinforced polymer (CFRP) systems.

Two of the damaged PC double-T girders from Bridge #56 were set aside for upgrade and testing. Each of the two members was saw cut in half longitudinally to provide a total of four 915 mm (36 in) wide by 12.2 m (40 ft) long single-T specimens. The specimens were 585 mm (23 in) deep, with a 125 mm (5 in) thick flange. They had four (4) rows of prestressing tendons, each
row consisting of a single 13 mm (0.5 in) diameter strand (see Figure 1) with an area of 98 mm² (0.153 sq.in). The strands were single point depressed at mid-span to a depth of 50 mm (2 in) from the bottom face. In addition, there were two layers of mild steel reinforcement running longitudinally and two layers running laterally. Mild steel reinforcement in the web consisted of single-legged 12 mm (#4) bars, positioned at the center of the web. These bars (not considered in the analysis as shear reinforcement) terminated 102 mm (4 in) from the bottom face and were spaced at approximately 255 mm (10 in) on center. Material properties based on coupons extracted from the girders or manufacturers’ datasheets were: \( f'_{c} = 46.2 \) MPa (6,700 psi); \( f_y \) (mild steel) = 420 MPa (60 ksi); \( f_{pu} \) (prestressing steel) = 1860 MPa (270 ksi); and \( E_s \) (prestressing steel) = 196.5 GPa (28.5 Msi).

1.2. Summary of Phase I
Three specimens were tested at the Civil Infrastructure Systems Testing Lab (CISL) at Kansas State University (Peterman and Reed 2002) with a 3-point bending configuration over a clear span of 11.6 m (38 ft) (Phase I) and one was set aside for the University of Missouri-Rolla where this study was completed (Phase II).

The first specimen, which was tested as the benchmark in Phase I, showed that the damaged girder was still able to withstand the original H-15 truck design load according to AASHTO Standards Specifications (AASHTO 1996). The second specimen was strengthened in flexure, using two plies of FRP laminate applied by manual lay-up to the bottom of the web and extending 95 mm (3.7 in) on each side of the web. Each ply had a fiber thickness of 0.16 mm (0.0065 in) and a total width of 305 mm (12 in) (Master Builders Technologies 1998). Finally, in order to prevent peeling of the laminate ends, a single ply U-wrap was installed at the flexural reinforcement terminations. Test results of this specimen showed that the upgrade scheme could increase the flexural capacity of the benchmark. However, due to shear deficiency, the full flexural capacity was not achieved and a premature horizontal shear failure at the level of prestressing steel occurred in the girder prior to FRP rupture.

A shear friction approach was used to design the FRP U-wrap for the third specimen that reached its full flexural capacity and failed due to flexural FRP rupture. The shear reinforcement consisted of double-ply FRP U-wraps 150 mm wide at 455 mm on centers (5 in at 18 in o.c.). Test results of Phase I are summarized in the first three rows of Table 1. A detailed explanations of the values listed in this table is deferred to the discussion section. The column labeled \( E_f A_f / E_s A_s \) provides the axial stiffness ratio, where \( E_f A_f \) is the axial stiffness (i.e., product of cross sectional area times elastic modulus) of the CFRP flexural reinforcement and \( E_s A_s \) is the axial stiffness of the steel prestressing tendons. The column labeled effective reinforcement represents the expression \( 2A_f E_f (\sin \alpha + \cos \alpha) / s \) (i.e., axial stiffness of the FRP shear reinforcement per unit length of girder) where \( \alpha \) is the slope of the fiber direction and \( s \) is the reinforcement spacing.

2. TEST PHASE II
One of the four single-T PC specimens was tested at the High-Bay Structures Laboratory of the University of Missouri-Rolla. The objective of this test was to investigate the effectiveness of the strengthening techniques based on CFRP rectangular bars installed as near surface mounted (NSM) reinforcement for shear upgrade combined with externally bonded pre-cured FRP laminate for flexural upgrade.

2.1. Flexural Strengthening
Flexural strengthening consisted of a pre-cured CFRP laminate 100 mm (4 in) wide and 1.4 mm (0.055 in) thick.
Table 1. Comparison of test results for Phases I and II (girder cross-section at 914 mm (3 ft) from midspan)

<table>
<thead>
<tr>
<th>Code</th>
<th>Test Setup</th>
<th>Failure Load kN (kips)</th>
<th>$E_s/A_s$</th>
<th>Max. Moment* kN (k-ft)</th>
<th>Moment** Increment (%)</th>
<th>Normal. Increment</th>
<th>Eff. Reinf.† kN/mm (kip/in)</th>
<th>Max Shear* kN (kips)</th>
<th>Shear‡ Increment (%)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>(Δ)</td>
<td>130.0 (29.2)</td>
<td></td>
<td>384.0 (283.5)</td>
<td>-</td>
<td>-</td>
<td>66.7 (15.0)</td>
<td>Flexure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>S2</td>
<td>(Δ)</td>
<td>160.0 (36.0)</td>
<td>0.15</td>
<td>458.2 (338.5)</td>
<td>19.3</td>
<td>129</td>
<td>-</td>
<td>81.8 (18.4)</td>
<td>Shear</td>
<td></td>
</tr>
<tr>
<td>S3</td>
<td>(Δ)</td>
<td>162.0 (36.5)</td>
<td>0.15</td>
<td>463.6 (342.5)</td>
<td>20.7</td>
<td>138</td>
<td>25.0 (14.3)</td>
<td>83.2 (18.7)</td>
<td>Flexure (FRP rupture)</td>
<td></td>
</tr>
<tr>
<td>S4</td>
<td>(Δ Δ)</td>
<td>210.0 (47.2)</td>
<td>0.38</td>
<td>437.0 (322.5)</td>
<td>13.8</td>
<td>36</td>
<td>52.0 (29.7)</td>
<td>125.5 (28.2)</td>
<td>53.4 (FRP delamination)</td>
<td></td>
</tr>
</tbody>
</table>

* Includes specimen self-weight
† Eff. Reinf. = $2A_f E_s (\sin \alpha + \cos \alpha)/s$
‡ Based on specimen S1
§ Based on specimen S2
(S&P 2003) applied on the bottom surface of the web, as depicted in Figure 2 and Figure 3. The mechanical properties of the laminate used for flexural strengthening are summarized in Table 2. The pre-cured CFRP laminate was produced by pultrusion, a continuous manufacturing process where fibers are impregnated with a thermoset resin and form a composite molded and cured through a heated die. A laminate produced by pultrusion typically has a fiber volume fraction of approximately 70%.

After concrete surface preparation by sandblasting, the installation procedure was simple and involved: 1) cutting the laminate to length; 2) applying the epoxy adhesive on the laminate; 3) pressing the laminate into the concrete surface; and 4) removing excess resin (see Figure 3). Finally, to prevent peeling of the FRP laminate ends, a single-ply CFRP wrap, 508 mm. (20 in) wide and 0.16 mm (0.0065 in) thick (fiber only) (Master Builders Technologies 1998) was placed at each end of the
longitudinal FRP laminate only partially over the girder web not to affect shear capacity. The properties of the CFRP U-wrap are reported in Table 2.

### 2.2. Shear Strengthening

The shear strengthening was attained with CFRP rectangular bars spaced at 200 mm (8 in) along the girder with an inclination of 60 degrees. Even though it is recognized that vertical bars would be easier to install, preference was given to inclined bars to improve the effectiveness of the shear strengthening. The pultruded rectangular FRP bars had dimensions 2 by 16 mm (0.079 by 0.63 in) with mechanical properties as listed in Table 2 (Hughes Brothers 2003). The groove cut into the concrete to receive the FRP bar had dimensions of 6 by 19 mm (0.25 by 0.75 in).

The advantages of using CFRP rectangular bars for structural strengthening installed with the NSM technique include:

- Surface preparation is minimized;
- Grooving is obtained with a single saw cut without any concrete chipping;
- After installation, the NSM bar is protected from mechanical damage and environmental deterioration;
- The quality of the concrete inside the groove is typically superior to the surface concrete.

The installation of the CFRP rectangular bars was performed according to the following sequence:

- Using a diamond blade concrete saw, a groove was cut;
- The groove was masked to prevent excess adhesive from staining the concrete, that would affect the aesthetic appearance;
- The groove was thoroughly cleaned using a vacuum and/or compressed air;
- The groove was half-filled with the epoxy-based adhesive. Care was taken to avoid entrapped air;
- The CFRP rectangular bar was placed on edge into the groove, as shown in Figure 4.
- Removal of excessive adhesive and general clean-up were the final steps of the procedure.

The shear strengthening was extended over the entire girder to simulate a field situation independently of the

![Figure 4. Installation of rectangular CFRP bar for shear strengthening](image)
Shear Strengthening of a PC Bridge Girder with NSM CFRP Rectangular Bars

2.3. Test Setup

The specimen was tested upside down under a four-point load configuration with a clear test span of 9.14 m (30 ft). The constant moment region was 1.82 m (6.0 ft) (see Figure 6). Such a test setup ensured more stability during the test and allowed a clear view of the FRP.

Two hydraulic jacks equipped with 890 kN (200 kips) load cells were used to apply and measure the load. Real time recording of structural response was achieved using an electronic data acquisition system. Two linear variable displacement transducers (LVDTs) were positioned (one on each side of the flange) at mid-span and two stringer-type LVDTs were placed close to the ends of the girder to measure deflection at the location of the jacks. Finally, 13 strain gages were applied on the FRP rectangular bars where maximum shear was expected (see Figure 7) two strain gages were applied on
the FRP laminates at 100 mm (4 in) from each side of the span centerline, and two strain gages were applied on the concrete top flange at the same positions (see Figure 8). The load was applied in cycles of load and unload as shown in Table 3.

### Table 3. Load-unload cycles for specimen S4

<table>
<thead>
<tr>
<th>Cycle</th>
<th>Load Range kN (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0-44.6-0 (0-10-0)</td>
</tr>
<tr>
<td>2</td>
<td>0-89.2-0 (0-20-0)</td>
</tr>
<tr>
<td>3</td>
<td>0-135.6-0 (0-30-0)</td>
</tr>
<tr>
<td>4</td>
<td>0-178.0-0 (0-40-0)</td>
</tr>
<tr>
<td>5</td>
<td>0-failure</td>
</tr>
</tbody>
</table>

**2.4. Experimental Results**

Partial debonding of the FRP laminate used for flexural reinforcement was observed at test load of 153 kN (34.6 kips) and a total debonding followed at an ultimate test load of 210 kN (47.2 kips) (see Figure 9). This corresponds to a maximum moment of 437 kN-m (322.5 kip-ft) and a maximum shear of 125.5 kN (28.2 kips) combining the effects of the applied test loads and the girder self-weight.

The maximum deflection at the location of the jack was 200 mm (8 in) (see Figure 10) while at mid-span it was 20 mm (0.8 in), so that a total deflection of 220 mm (8.8 in) was experienced at ultimate. The highest recorded level of strain on the flexural CFRP laminate before failure was 12,000 µε, as shown in Figure 11. Finally,
the highest recorded strain on the rectangular CFRP bars due to shear, which corresponded to the ultimate load, was approximately 6,500 $\mu$e. (see Figure 12).

3. DISCUSSION

The objective of the proposed investigation was to evaluate the shear and flexural improvement provided by CFRP NSM rectangular bars and the pre-cured laminate, respectively. In order to perform a comparison between the test results of Phases I and II, the maximum shear and bending moment obtained in the three specimens of Phase I (namely specimens S1, S2, and S3) were recomputed at a cross-section at 0.9 m (3 ft) from mid-span. This was the location where maximum shear and bending moment were obtained in the testing setup of specimen S4.
3.1. Flexural Evaluation

In order to evaluate the flexural capacity improvement given by the pre-cured FRP laminate, the nominal moment capacity of specimen S4 was compared with the maximum moment recorded for specimen S1 at the same cross-section. The control specimen S1 exhibited a flexure-controlled failure mode. Considering the sum of both applied load and self-weight, the ultimate moment of specimen S4 at 0.91 m (3 ft) from mid-span was 437 kN-m (322.5 kip-ft), whereas at the same cross-section, the ultimate moment of the control specimen S1 was 384 kN-m (283.5 kip-ft). Thus, the use of the FRP laminate increased the moment capacity by 13.8 % as compared to the control specimen. For specimens S2 and S3, the maximum moments (at 0.91 m (3 ft) from mid-span) when failure occurred, were 458.2 kN-m (338.5 kip-ft) and 463.6 kN-m (342.5 kip-ft), providing capacity increments of 19.3 and 20.7%, respectively. These results are summarized in Table 1. The normalized increment shown in the table is calculated by dividing the moment increment by the axial stiffness ratio (i.e., flexural CFRP laminate over the flexural steel reinforcement). This parameter provides a measure of the efficiency of the different systems. Its validity is...
somehow relative, because some of the failure modes of the four girders were not always flexure-controlled, but nonetheless it is indicative of the benefits of the FRP system in each test.

3.2. Shear Evaluation

Considering both applied test load and self-weight, the maximum shear in specimen S4, when flexural failure occurred, was 125.5 kN (28.2 kips). The ultimate shear in S2, that had a shear-controlled failure, was 81.8 kN (18.4 kips). Thus, specimen S4 showed an improvement in shear capacity of at least 53%, over this base strength. This value may be considered as a lower bound, since specimen S4 failed due to flexural FRP laminate debonding, that did not allow achieving full shear capacity. Finally, specimen S3, that also experienced a flexure-controlled failure, showed a maximum shear value of 83.2 kN (18.7 kips). A summary of the shear results is shown in Table 1. In this table, the shear capacity increment can be related to the effective axial stiffness of the shear strengthening reinforcement expressed as $2AE_f/(\sin \alpha + \cos \alpha)/s$.

4. MODELING CONSIDERATIONS FOR SHEAR

The approach used to calculate the nominal shear capacity of a member strengthened using NSM bars is similar to that used in ACI 440 (2002) for the case of externally bonded FRP laminates. Eqn 1 is applicable to NSM systems:

$$V_n = V_c + V_r + V_f$$

Several parameters influence the NSM FRP bars contribution ($V_f$) to the shear capacity, such as quality of bond, FRP bar type, groove dimensions, and quality of substrate material. When computing $V_f$, two strain limits need to be taken into account (De Lorenzis and Nanni 2001a), namely: maximum strain from bond-controlled failure, and maximum strain threshold of 0.004. The latter is suggested to maintain the shear integrity of the concrete (Khalifa et al. 1998), and to avoid large shear cracks that could compromise the aggregate interlock mechanism.

The shear strength provided by the NSM reinforcement applied symmetrically on both sides of the girder web can be determined by calculating the force resulting from the tensile stress in the FRP bars across the assumed crack, and it is expressed for rectangular bars by Eqn 2:

$$V_f = 4(a + b)\tau_b L_{tot}$$

where $a$ and $b$ represent the cross-sectional dimension of the rectangular FRP bar, and $\tau_b$ represents the average bond stress of the bars crossed by a shear crack. Experimental data available on 10-mm (#3) carbon FRP deformed bars demonstrate that when using an epoxy-based resin in a groove size at least 1.5 times the bar diameter, a conservative value of $\tau_b = 6.9$ MPa (1.0 ksi) can be used (De Lorenzis and Nanni 2001b). $L_{tot}$ can be expressed as $L_{tot} = \sum L_i$, where $L_i$ represents the length of each single NSM bar intercepted by a shear crack (Figure 13) expressed as:

$$L_i = \begin{cases} \frac{s}{\cos \alpha + \sin \alpha} i \leq l_{0.004} & i = 1 \ldots \frac{n}{2} \\ \ell_{net} - \frac{s}{\cos \alpha + \sin \alpha} i \leq l_{0.004} & i = \frac{n}{2} + 1 \ldots n \end{cases}$$

where $\alpha$ represents the slope of the FRP bar with respect to the longitudinal axis of the beam, $s$ is the horizontal FRP bar spacing, and $\ell_{net}$, defined as follows:

$$\ell_{net} = \ell_h - \frac{2c}{\sin \alpha}$$

![Figure 13. Sketch showing definition of $L_i$ for Specimen S4](image)
represents the net length of a FRP bar as shown in Figure 14 to account for cracking of the concrete cover and installation tolerances. In Eqn 4, \( l_b \) is the actual length of a FRP bar, and \( c \) is the clear concrete cover of the internal longitudinal reinforcement.

The first limitation in Eqn 3, \( s \cdot i/\left(\cos \alpha + \sin \alpha\right) \) and \( \ell_{\text{net}} - s \cdot i/\left(\cos \alpha + \sin \alpha\right) \) takes into account bond as the controlling failure mechanism, and represents the minimum effective length of a FRP bar intercepted by a shear crack as a function of the term \( n \):

\[
n = \frac{\ell_{\text{eff}}(1 + \cot \alpha)}{s}
\]  

(5)

where \( n \) is rounded off to the lowest integer (e.g., \( n = 10.7 \Rightarrow n = 10 \)), and \( \ell_{\text{eff}} \) represents the vertical length of \( \ell_{\text{net}} \) as shown in Figure 14 written as follows:

\[
\ell_{\text{eff}} = \ell_b \sin \alpha - 2c
\]

(6)

The second limitation in Eqn 3, \( l_{0.004} \), takes into account the shear integrity of the concrete by limiting at 0.004 the maximum strain in the FRP reinforcement. From the force equilibrium condition and for rectangular bars, \( l_{0.004} \) can be determined as follows:

\[
l_{0.004} = 0.002 \frac{a \cdot b \cdot E_f}{a + b} \tau_b
\]

(7)

where \( E_f \) represents the elastic modulus of the FRP bar.

From Eqns 4 to 7 and for Specimen S4 one can obtain:

\[
\ell_{\text{net}} = \ell_b - \frac{2c}{\sin \alpha} = 20'' - \frac{2(1'')}{\sin(60^\circ)} = 17.7''
\]

\[
\ell_{\text{eff}} = \ell_b \sin \alpha - 2c = (20'') \sin(60^\circ) - 2(1'') = 15.3''
\]

\[
n = \frac{\ell_{\text{eff}}(1 + \cot \alpha)}{s} = \frac{(15.3'')[1 + \cot(60^\circ)]}{8''} = 3.021 \Rightarrow 3
\]

\[
l_{0.004} = 0.002 \frac{a \cdot b \cdot E_f}{a + b} \tau_b = 0.002 \frac{(0.079'')(0.63'')(18,000 \text{ ksi})}{(0.079'') + (0.63'')(1.0 \text{ ksi})} = 2.5''
\]

(8)

Finally, by applying Eqn 3 the following lengths are recognized:

\[
L_1 = \frac{8''}{\cos(60^\circ) + \sin(60^\circ)} = 5.8'' > l_{0.004} = 2.5'' \Rightarrow L_1 = 2.5''
\]

\[
L_2 = 17.7'' - \frac{8''}{\cos(60^\circ) + \sin(60^\circ)} = 6.1'' > l_{0.004} = 2.5'' \Rightarrow L_2 = 2.5''
\]

\[
L_3 = 17.7'' - \frac{8''}{\cos(60^\circ) + \sin(60^\circ)} = 0.1'' < l_{0.004} = 2.5'' \Rightarrow L_3 = 0.1''
\]

\[
L_{\text{tot}} = L_1 + L_2 + L_3 = 5.1''
\]

(9)

The FRP contribution to the shear capacity, \( V_f \), is given by Eqn 2:

\[
V_f = 4(a + b)\tau_b L_{\text{tot}} = 4(0.079'' + 0.63'')(1.0 \text{ ksi})(5.1'') = 14.5 \text{ kip}
\]

(10)

Based on conventional PC theory and the proposed model, the nominal shear capacity of the girder prior to and after strengthening were computed and compared to the experimental values as shown in Table 4. The reported data show a good match, in fact: the ratio \( V_n/V_u \) is lower than 1.0 for the case of specimen S2 that failed

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Figure 14. Definition of \( \ell_{\text{net}} \) and \( \ell_{\text{eff}} \)
in shear (i.e., analysis is slightly conservative) and it is higher than 1.0 for the case of Specimen S4 that failed in flexure (i.e., shear capacity is not reached).

5. CONCLUSIONS

Overall, this research allowed comparing and checking the effectiveness of numerous FRP upgrade systems with reference to both shear and flexural strengthening. The results of Phase I show that CFRP laminates applied on the bottom surface of the specimens are definitely valid for flexural upgrade, while Phase II shows that CFRP rectangular bars based on the NSM technique represents an innovative and effective system for shear strengthening. The latter is confirmed by high strain values recorded by strain gages applied on CFRP bars during the test. For what concerns the flexural strengthening, the precured laminates tested in Phase II appeared to provide lower performance when compared to laminates installed by manual lay-up. Furthermore, a comparison between the results provided by Phase I and II underlines that the specimen failure mode varies depending on the selected upgrade system.

Further investigations are necessary in order to optimize the combination of flexural and shear upgrade and in particular to prevent debonding of the flexural strengthening.

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Table 4. Comparison of analytical and experimental results

<table>
<thead>
<tr>
<th>Code</th>
<th>Test Setup</th>
<th>$V_c$ kN (kips)</th>
<th>$V_s$ kN (kips)</th>
<th>$V_f$ kN (kips)</th>
<th>$V_n$ kN (kips)</th>
<th>Max Shear $V_u$ kN (kips)</th>
<th>$V_n/V_u$</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2</td>
<td>$\Delta$</td>
<td>65.4 (14.7)</td>
<td>0</td>
<td>0</td>
<td>65.4 (14.7)</td>
<td>81.8 (18.4)</td>
<td>0.80</td>
<td>Shear</td>
</tr>
<tr>
<td>S4</td>
<td>$\Delta \Delta$</td>
<td>65.4 (14.7)</td>
<td>0</td>
<td>64.5 (14.5)</td>
<td>129.9 (29.2)</td>
<td>125.4 (28.2)</td>
<td>1.04</td>
<td>Flexure</td>
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<tr>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>(FRP delamination)</td>
</tr>
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</table>

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Antonio Nanni is the V & M Jones Professor of Civil Engineering at University of Missouri-Rolla. He is an active member in the technical committees of ACI (Fellow), ASCE (Fellow), ASTM and TMS. He was the founding Chairman of ACI Committee 440 - FRP Reinforcement and is the current Chairman of ACI Committee 437 – Strength Evaluation of Existing Concrete Structures. Dr. Nanni is the Editor-in-Chief of the ASCE Journal of Materials in Civil Engineering.

Marco Di Ludovico is a Graduate Assistant at the University of Naples - Federico II, Naples, Italy. He was a visiting student at the University of Missouri-Rolla and participated in the Socrates Intensive Programme “FRP Reinforcement for Concrete,” held at Ghent University, Ghent, Belgium.

Renato Parretti is a senior structural engineer with Co-Force America, Rolla, MO responsible for numerous FRP design projects throughout the world. He is a member of ACI and ASCE and serves as an active member on ACI Committee 437, Strength Evaluation of Existing Concrete Buildings, and ACI Committee 440, Fiber Reinforced Polymer Reinforcement. Mr. Parretti holds a B.S. in Civil Engineering from the University of Florence, Italy. He is a registered PE in Italy.