Design Construction and Field Validation of the Southview Bridge in Rolla, Missouri

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ABSTRACT: A research project was undertaken to evaluate the use of post-tensioned FRP for bridge deck construction. The type of structure selected for this project is a four-span continuous concrete slab having GFRP bars for top and bottom mats and CFRP reinforcement for internal post-tensioning of the bridge deck. This bridge is located in Rolla, Missouri. One lane of the bridge was already built using a conventional four cell steel reinforced concrete box culvert. One lane and sidewalk needed to be added. This additional lane was constructed using FRP bars as internal reinforcement. This study included the design of the FRP portion of the bridge using existing codes when appropriate, validation of the FRP technology through a pre-construction investigation conducted on two specimens representing a deck strip, the construction of the bridge and a field validation through load test. The results showed that a combination of post-tensioned and mild FRP reinforcement results in an economical solution for a deck system with low deflection and high shear strength at a minimum deck thickness.

INTRODUCTION

For more than 20 years, the Federal Highway Administration (FHWA) of the United States Department of Transportation has researched and demonstrated the use of FRP composites, space age technology for bridge construction. In particular, the use of FRP bars as internal reinforcement for concrete has been increasing steadily over the last several years. Fiber-reinforced Polymer (FRP) has certain advantages over steel, such as high tensile strength, light weight, and corrosive resistance. These advantages make it an ideal alternative reinforcement (ACI 440.1R-03; Nanni, 2001). Studies on the short (Abdelrahman et al., 1995) and long (Zou, 2003) -term behavior of concrete beams prestressed with FRP tendons showed the high potential of Carbon FRP (CFRP) bars per replacing steel as post-tensioning material.

The interest in the use of FRP composites in prestressed concrete is mainly based on durability issues. Corrosion of prestressing steel tendons can cause serious deterioration of infrastructure. Properties like high tensile strength and high resistance to corrosion make FRP composites good candidates for prestressed and post-tensioned tendons (Fico et al., 2005).

The objectives of this research project were as follows:

1. Evaluate the feasibility, behavior and effectiveness of an innovative deck system, showing how glass and carbon FRP (GFRP and CFRP), in the form of passive and active internal reinforcement, could be a solution replacing steel reinforcement. The design also accounted for the fact that, unlike steel reinforced concrete sections, members reinforced with FRP bars have relatively small stiffness after cracking. Therefore, serviceability requirements have also been examined.

2. Validate the proposed combination of FRP technologies in the field via load testing.

PROJECT DESCRIPTION

The City of Rolla, MO has made available a bridge (Southview Drive on Carter Creek) to demonstrate the use of FRP bars and tendons in new constructions. One lane of the bridge was already constructed using conventional four-cell steel RC box culvert. It consists of a steel RC slab about 0.25 m thick, as depicted in Figure 1.

Figure 1 – Original Construction

The slab deck is continuous over three intermediate RC vertical walls, and the overall length of the bridge is...
roughly 12 m. The new deck was built on three conventional RC walls as for the existing structure. The construction of the FRP reinforced slab, plus a 2 m wide conventional RC sidewalk on the opposite side, allowed extending the overall width of the bridge from 3.9 m to 11.9 m, as Figure 2 shows. The construction of the bridge started on July 2004 and finished on October 2004.

Figure 2 – New Cross Section of the Bridge

**BRIDGE DESIGN**

The analysis and design of the FRP concrete bridge deck was based on the following assumptions:

- Nominal properties for FRP reinforcing material taken from the manufacturer published data and considered as initial guaranteed values to be further reduced to take into account the environmental reduction factors as given in ACI 440.1R-03;
- Load configurations consistent with AASHTO Specifications;
- Design carried out according to ACI 440.1R-03; and
- Effects due to the skew neglected.

The design properties of concrete and FRP bars used are summarized in Table 1.

### Table 1 – FRP Design Material Properties

<table>
<thead>
<tr>
<th>Concrete Compressive Strength f′c (MPa)</th>
<th>FRP Internal Reinforcement Type</th>
<th>FRP Bar Size (mm)</th>
<th>FRP Tensile Strength f′u (MPa)</th>
<th>FRP Tensile Strain εu</th>
<th>FRP Modulus of Elasticity E (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>41.4 (Deck)</td>
<td>GFRP</td>
<td>φ9</td>
<td>758</td>
<td>0.018</td>
<td>40.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>φ13</td>
<td>689</td>
<td>0.017</td>
<td>40.8</td>
</tr>
<tr>
<td></td>
<td>CFRP</td>
<td>φ9</td>
<td>2068</td>
<td>0.017</td>
<td>124.1</td>
</tr>
</tbody>
</table>

Bending moments and shear forces are summarized in Table 2. Columns (1) and (3) represent the factors to be applied to dead and live load, respectively. Columns (2) and (4) show unfactored moment and shear, due to dead and live load, respectively.

The flexural design of a FRP RC member is similar to the design of a steel RC member. The main difference is that both concrete crushing and FRP rupture are potential mechanisms of failure. Assuming the post-tensioned tendons strained at 65% of their ultimate capacity and a 35% of total losses, the theoretical moment capacity is found to be 64 kN-m. If no post-tensioning is provided, the corresponding moment capacity would be just the same, being the strain in the GFRP the controlling one in the strain diagram both in presence of post-tension and not.

### Table 2 – Moment and Shear per Unit Strip (Live and Dead Load)

<table>
<thead>
<tr>
<th>Loading Conditions</th>
<th>Moment and Shear</th>
<th>Dead Load</th>
<th>Live Load</th>
<th>Ultimate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Truck</td>
<td>M[kN-m/m]</td>
<td>6.27</td>
<td>29.69</td>
<td>92.09</td>
</tr>
<tr>
<td></td>
<td>M[kN-m/m]</td>
<td>6.27</td>
<td>23.52</td>
<td>77.56</td>
</tr>
<tr>
<td></td>
<td>Shear [kN/m]</td>
<td>4.92</td>
<td>15.42</td>
<td>160.53</td>
</tr>
<tr>
<td></td>
<td>Shear [kN/m]</td>
<td>4.92</td>
<td>17.29</td>
<td>59.88</td>
</tr>
<tr>
<td></td>
<td>Shear [kN/m]</td>
<td>4.92</td>
<td>12.47</td>
<td>134.25</td>
</tr>
</tbody>
</table>

The concrete contribution to the shear capacity, of the bridge deck was calculated according to ACI 440.1R-03 and ACI318-99, sec. 11.4 in order to account for the positive effect of the post-tensioning. The factored shear capacity is \( \phi V_u = \phi V_{c,f} = 162 kN/m \) which is higher than the shear demand \( V_u = 160.5 kN/m \). If the post-tensioning action was not considered, the shear capacity would have been \( \phi V_u = \phi V_{c,f} = 95 kN/m \) which is much smaller than the demand (the shear capacities have been computed from an analysis performed on the cross section flush with the vertical wall representing the support).

The amount of FRP reinforcement was determined in order to satisfy the service conditions in terms of crack width, long-term deflection, creep rupture and fatigue. Again, such quantities were computed according to ACI 440.R-03.

Additional details on the design procedures and equations can be found in Fico et al., 2005.

**PRE-CONSTRUCTION INVESTIGATION**

### SPECIMENS PREPARATION

Two identical specimens representing a deck strip 457 mm wide, 254 mm deep and 7 m long, were fabricated and tested as continuous slabs over three supports. One specimen was used to investigate the flexural behavior (“flexural” specimen), while the other one the shear behavior (“shear specimen”). The testing allowed validating the design calculations both in terms of flexure and shear capacities.

The specimens had the same reinforcement percentage of the bridge deck: they were reinforced using 3 φ19 GFRP bars as top and bottom mat and 2 φ9 CFRP bars as post-tensioning tendons. The position of the tendons was varied along the specimen in order to match the moment demand. In addition, in order to reproduce the actual field
conditions, also $\phi_3$ GFRP bars spaced 305 mm on center were placed in the transverse direction Figure 3 shows a detailed layout of the specimens’ reinforcement, while Figure 4 shows the position of the CFRP tendons.

![Figure 3 – Reinforcement Layout (All Dim. in mm)](image)

The post-tensioning of the tendons was executed 28 days after the pouring of the concrete. The CFRP bars were post-tensioning by applying a force of 98 kN (22 kip) using hydraulic jacks at both ends. This level of post-tensioning corresponds to 65% of the ultimate capacity of the CFRP bars. Such pre-stressing level was chosen in order to respect, after the initial strain losses, the creep rupture limits dictated for CFRP bars according to ACI 440.1R-03, Sec. 9.2.

Initially the post-tensioning load was applied to only one end of the slab causing the breaking of the FRP tendons due to the high eccentricity of the active reinforcement and to the friction between ductwork and tendons. This problem was solved by applying the post-tensioning load in steps of 31 kN from both ends by mean of two hydraulic jacks. The loading rate was the same for the two spans until the forces were generated using two electrical strain gages attached on the bar. This solution was suitable because the increased losses after the release of the tendons induced by the new pre-stressing system (30%) were less than the ones assumed for design (35%).

The steel wedge anchorage system used to anchor the CFRP bar and to react against the hydraulic jack was a resin-free three part system developed at the University of Waterloo, Canada. It included an outer steel cylinder; a four-piece wedge and an inner sleeve (see Figure 4). The inner sleeve is made out of copper/steel and it is deformable. The four-piece wedge was placed evenly around the inner sleeve and inserted into the outer steel cylinder. The anchorage system was later secured by tapping the inner sleeve and four-piece wedge into the outer steel cylinder with a hammer.

![Figure 4 – Steel Wedge Anchorage System](image)

**TEST SETUP**

Each slab was tested as a continuous member on three supports, comprising of a 3.6 m and 1.8 m long span. The positions of the two loading points were chosen such to force flexural and shear failure for the flexural and shear specimens respectively. They were placed at the mid-spans for the flexural specimen and 0.9 m away from the central support in the case of the shear specimen (See Figure 5).

![Figure 5 – Test Setup (All Dimensions in mm)](image)

**The flexural specimen (See Figure 5–a) was instrumented using three load cells:** two of them were placed under the loading points while the third one was placed under one of the supports in order to determine the end reaction and therefore the real distribution of moments in the slab. Loads were applied to 102 x 457 x 25 mm steel plates resisting on the slab. The loads were generated by means of 30 ton hydraulic jacks reacting against a steel frame. The loading rate was the same for the two spans until
reaching 85 kN. After that, the load in the shorter span was kept constant while the one in the longer one was increased until failure. This solution was adopted in order to avoid shear failure at the central support. Linear Variable Displacement Transducers (LVDTs) were positioned at the loading points (two for each loading point) and at the supports in order to record maximum displacements and support settlements. A total of 21 Electrical Strain Gages (ESG) were used to monitor the strain at the most critical sections. They were placed on each GFRP bar in correspondence with loading points and the central support. In addition, at the same locations, an additional strain gage was attached on the compressive face of the slab in order to have an additional backup point while determining the experimental moment-curvature response of the slab.

For the shear specimen, the number of load cells was reduced to two. Loads were applied to 102 x 457 x 25 mm steel plates resisting on the slab by means of a 100 ton hydraulic jack (See Figure 5–b). Two additional LVDTs were inserted in order to also measure the maximum displacement which, in this case, was not at the loading points.

The load was applied in cycles of loading and unloading. An initial cycle for a low load was performed on every specimen to verify that both the mechanical and electronic equipment was working properly.

TEST RESULTS AND DISCUSSION

For the “flexure” specimen, the first flexural crack was observed on the longer span when the load was approximately 66 kN on both spans. As the loads were increased, some of the cracks started to extend diagonally to form shear cracks. At maximum loads, 163 kN and 100 kN for long and short spans respectively, concrete crushing at the top was observed, indicating flexural failure. This was immediately followed by a sudden shear failure which also caused the rupture of the CFRP bars due to kinking (See Figure 6-a).

The first crack in the “shear” specimen was observed at a load approximately of 89 kN in correspondence of the central support where the moment was maximum. As the load was increased, the newly formed cracks between the central support and the loading point on the central span started to extend diagonally to form a shear crack. The failure of the specimen occurred for an applied load equal to 273 kN due to diagonal tension shear.

Table 3 compares the experimental results with the theoretical moment and shear capacities of the slabs computed according to ACI 440.1R-03. For the same specimen the Tureyen A. K. and Frosh R. J. approach overestimated the shear capacity because using the neutral axis at the service conditions may not be correct when the member is approaching its flexural capacity.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Experimental</th>
<th>Theoretical</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maximum Bending Moment (kN-m)</td>
<td>Maximum Shear Force (kN)</td>
</tr>
<tr>
<td>Flexure</td>
<td>148.9</td>
<td>81.4</td>
</tr>
<tr>
<td>Shear</td>
<td>70.2</td>
<td>142.3</td>
</tr>
</tbody>
</table>

The shear specimen presented a shear capacity higher than the theoretical demonstrating again the safe approach of ACI 440.1R-03. For this specimen the Tureyen A. K. and Frosh R. J., approach was safe because the maximum bending moment experienced in the test was only half of the ultimate moment capacity determined from the “flexure” specimen.

Additional details on the pre-construction investigation and analysis can be found in Galati et al. 2004.

BRIDGE DECK CONSTRUCTION

The bridge addition construction was started in July 2004 and completed in October 2004. The following sections describe the different phases of the construction, problems occurred and adopted solutions.
SUBSTRUCTURE
The erection of the substructure and the extension of the existing abutments and walls were performed first. GFRP bars were inserted into the central wall in order to create a connection between the slab and the substructure (see Figure 7).

![Figure 7 – Construction Substructure](image)

DECK
After laying the slab formwork, 13 mm thick neoprene pads were placed on the two abutments and on the two walls that did not have the GFRP anchoring rebars, in order to avoid restraining horizontally the slab and therefore to effectively post-tension it. Hence the bottom and the top layers of GFRP mild reinforcement were placed.
Since the bridge presented a high skew (about 45°), a wooden board was built to make the slab end perpendicular to the tendons. This operation was performed in order to ease the post-tensioning phase. This was followed by the placing of plastic ducts to house the CFRP tendons. Such ducts were tied to the GFRP bars (see Figure 8).

![Figure 8 – Laying of FRP Reinforcement](image)

T connectors were used on each end of ducts in order to allow the injection of the grout after the pulling of the tendons. Strain gages were also attached on the longitudinal GFRP bars in order to monitor the slab during the service life.
A week after the curing of the slab, the CFRP tendons were post-tensioned. The pulling was achieved by means of the jack already used for the specimens.

Figure 9 shows the pulling device. It comprises an open steel box having enough room to push the wedges inside the chuck after pulling the tendon, a hydraulic jack to apply the pulling force, a round steel plate, a load cell to measure the load, a second plate and an outer chuck.

![Figure 9 – FRP Post-Tensioning Device (Fico et al., 2005)](image)

The tendons were pulled at both terminations by means of two hydraulic jacks, connected to two pumps using load steps of 15-20 kN per side. The applied load was measured using a data acquisition system connected to a computer allowing monitoring the applied load in real time. After reaching the desired load the wedges were pushed inside the inner chuck, so each jack was released, engaging in such way the inner chucks. At this point FRP tendon was cut with an electric saw.
The grout injection followed, after sealing the chucks to avoid grout leaking. After 4 days of curing the inner chucks were removed. Figure 15 shows a picture of the bridge after completion.

![Figure 10 – FRP Post-Tensioning Device(Fico et al., 2005)](image)

Finally, a barrier with GFRP reinforcement was built on the new “FRP side” (see Figure 11), in order to have a comparison during time with the steel reinforced concrete barrier on the opposite side, which was also built. This solution will contribute to show the increased durability of a bridge deck using FRP materials as reinforcement, mainly due to the absence of corrosion.
Shortly after construction of the bridge, the behavior of the bridge under load was examined. A picture of the bridge during the load test is shown in Figure 12.

Instrumentation utilized during the testing included direct current variable transformer (DCVT) transducers installed underneath the bridge to monitor deflection and electrical strain gages bonded on the concrete surface in the direction of the traffic. The strain gages installed on the FRP bars during construction did not work at the time of the testing. The location of the sensors is illustrated in Figure 13, with the symbol denoting each individual instrument.

Eight DCVT transducers were located at mid-span between four consecutive supports. No sensors were installed on the fourth span since the cables were not long enough to reach it. The two strain gages were installed in correspondence of DCVT 1 and DCVT 3.

Loading of the bridge was accomplished with a loaded dump truck placed at various locations on the bridge. Figure 14 shows the truck’s geometry and load per axle. The total weight of the truck was 241 kN with 148 kN and 93 kN on each of the two axles from the front to the rear of the truck, respectively. Although dump truck and HS20 trucks differ in their geometry, the loading configuration that maximize the stresses and deflections at mid span could still be accomplished.

Two passes of the truck were made, each at a different transverse position on the bridge as showed in Figure 15. During each pass the truck was stopped at four longitudinal locations corresponding to the middle section of each span as showed in Figure 15. During each stop, the truck stationed for at least two minutes before proceeding to the next location in order to allow stable readings.

The bridge performed well in terms of overall deflection. In fact, the maximum deflection measured during the load test was 0.35 mm which is below the allowable deflection prescribed by AASHTO, 2002 Section 8.9.3 ($\delta_{max} \leq L/800 = 3.8$ mm). Such small displacements of the bridge were explained with the arching action occurring due to the short spans and considerable thickness.
To validate the data obtained from the load tests, a linear elastic FEM analysis was conducted. For this purpose a commercially available finite element program SAP2000 was used.

Solid elements were chosen to model the concrete. The solid element is a brick element defined by eight nodes having three degrees of freedom at each node. For this project, the material properties of concrete were assumed to be isotropic and linear elastic, since the applied load was relatively low. The modulus of elasticity of the concrete was based on the measured compressive strength of the cylinders obtained at the pouring of the slab according to the standard equation ACI 318-05 Section 8.5.1:

\[ E_c = 57000 \sqrt{f_c} \approx 24.8 \text{ GPa} \]

Each element was meshed to be 89 mm × 127 mm × 152 mm. The bridge was modeled as hinged in correspondence of each supporting wall. The load was applied on 8 nodes simulating the truck wheels. A picture of the finite element model is showed in Figure 16.

\[ \text{Figure 16 – FEM Model Geometry} \]

The experimental and analytical results for pass 2 in the transversal direction are reported in Figure 17. The graph shows the good match in deflection between the experimental and analytical results, therefore it can be used to assess the performance and the load rating of the bridge over time.

\[ \text{Figure 17 – Comparison Between Experimental and Theoretical Results} \]

CONCLUSIONS

The following conclusions can be drawn:
1. The post-tensioning allowed increasing the shear capacity of the slab by more than 70%. Moreover serviceability requirements are fully satisfied.
2. Utilizing FRP in the form of reinforcing bars allows for the use of many steel-RC concrete practices. The fabrication and installation details were nearly identical to the methods regularly utilized for steel-reinforced slabs.
3. The load testing of the bridge allowed verifying its safety. The developed numerical model showed a good match with the experimental results, therefore it can be used to assess the performance and the load rating of the bridge over time.
ACKNOWLEDGMENTS

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