DESIGN AND CONSTRUCTION OF A BRIDGE DECK USING FRP AS MILD AND POST-TENSIONED REINFORCEMENT

Relatore: Ch.mo Prof. Ing. G. Manfredi

Correlatore: Ch.mo Prof. Ing. A. Nanni

Candidato: Raffaello Fico

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BRIDGE DECK CONSTRUCTION WITH INTERNAL FRP REINFORCEMENT

VALIDATION OF FRP COMPOSITE TECHNOLOGY THROUGH FIELD TESTING

Construction of Southview Bridge Deck

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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 (Southview Drive on Carter Creek). 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.

The combination of prestressed and non-prestressed FRP reinforcement resulted in an economical solution for a deck system with low deflection and high shear strength at a minimum deck thickness.

This study includes the design of the FRP portion of the bridge using existing codes when appropriate, the validation of the FRP technology through a pre-construction investigation conducted on two specimens representing a deck strip, construction and field evaluation through load testing of the bridge.

The results showed how FRP, in the form of GFRP as passive and CFRP bars as active internal reinforcement, could be a feasible solution replacing the steel reinforcement of concrete slab bridges, and specifically the enhanced shear capacity of the slab due to the CFRP prestressing.
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1 INTRODUCTION

1.1 Background

Reinforced and prestressed concrete (PC and RC) structures are facing a worldwide problem, which is the corrosion of the steel as a result of aging and aggressive environments. Steel corrosion leads to member degradation, endangers structural integrity and may even cause catastrophic failures. Research has been carried out in an effort to find the solution for this problem. The recent advancements in the field of material science have resulted in the development of new products that can be used in many areas of civil engineering where conventional materials have failed to provide satisfactory service life. Fiber Reinforced Polymers (FRP) have been proposed for use in lieu of steel for reinforcement and prestressing tendons in concrete structures. The promise of FRP materials lies in their high-strength, lightweight and non-corrosive, non-conducting, and non-magnetic properties. In addition, FRP manufacturing offers a unique opportunity for the development of shapes and forms that would be difficult or impossible with conventional steel materials. They can be manufactured as reinforcing bars and tendons for RC and PC structures, sheets and laminates for external strengthening of beams, slabs and masonry walls, wraps and shells for confinement of columns, etc.

The interest in the use of FRP composites in prestressed concrete is mainly based on durability issues. Corrosion of prestressing steel
tendons caused serious deterioration of infrastructure. Properties like high tensile strength and high resistance to corrosion would appear to make FRP composites good candidates for prestressing tendons.

1.2 **Research Objectives**

The objectives of the project at the University of Missouri Rolla were as follows:

1. Evaluate the feasibility, behavior, and effectiveness of the new deck system, showing how FRP, in the form of GFRP as passive and CFRP bars as active internal reinforcement, could be an excellent solution replacing the steel.
2. Provide analytical data in support of the enhanced shear capacity of the concrete slab due to the CFRP prestressing.

1.3 **Description of the project**

The City of Rolla in Missouri 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 reinforced concrete (RC) box culvert. It consists of a steel RC slab about 25 cm (10 in) thick, as depicted in Fig. 1. The slab deck is continuous over three intermediate reinforced concrete vertical walls, and the overall length of the bridge is roughly 12 m (40 ft). The new deck was built on three conventional RC walls as for the existing structure. The expansion phase included the removal of the curb from the existing RC deck to allow extending the overall width of the bridge from 3.9 m (12.8 ft) to
11.9 m (39 ft). The curb-to-curb width of the resulting bridge is 9.1 m (30 ft). The two additions consist of a FRP prestressed/reinforced concrete deck and a steel RC deck as shown in Fig. 2. The construction of the bridge started on July 2004 and finished on October 2004.

![Fig. 1 - Views of the Former Bridge](image)

![Fig. 2 - Cross Section of the Bridge After the Expansion](image)

### 1.4 Thesis outline

This thesis consists of six sections:

- Section one gives a brief introduction of the background of FRP applications in civil engineering and introduces the objectives of the research.
- Section two contains informations on existing prestressing and non-prestressing FRP bridge decks, so that they can be compared with the new deck system that is the subject of this
thesis. It also gives a summary of the main research works on shear behavior of prestressed FRP.

- Section three presents the pre-construction investigations conducted on two specimens representing a deck strip 457 mm (18 in) wide and 7 m (23 ft) long, fabricated and tested. The specimens were constructed by the contractor peculiar for the project allowing for his familiarization with the use of non-conventional materials. The testing of the specimens as continuous slabs over three supports allowed the validation of the design calculations in terms of flexure and shear capacities.

- Section four focuses on Southview Brigde design, providing the structural analysis of the new FRP concrete bridge deck based on the AASHTO HS20-44 design truck and providing calculations for its design using a combination of non-traditional corrosion-resistant composites materials.

- Section five details the installation of Southview bridge deck, focusing on the post-tensioning of the slab, the most considerable and crucial part of the project.

- Section six gives a brief summary of the steps developed in the previous sections and deepens the results of the research work and installation of the bridge-deck.
2 LITERATURE REVIEW

2.1 General

There are only about fifty bridges in the world with Fiber Reinforced Polymer superstructure as reported by the U.S. Department of Transportation Federal Highway Administration as of July 2002. The use of FRP bridge decks has been a direct result of technology transfer initiatives taken by the defense industry in late 1980’s and early 1990’s. In fact, many of the manufacturers of FRP bridge decks were directly or indirectly related to the aerospace composites industry. Furthermore, in recent years many state transportation departments in partnership with the Federal Highway Administration (FHWA) are introducing these new materials to bridge structures with the intention of gaining design and construction experience and long-term performance data on FRP, thus FRP deck systems are emerging as a viable alternative to conventional systems, namely reinforced-concrete slabs. Moreover the research on prestressing with FRP tendons is getting attention mainly due to the fact that nearly half of the nation’s bridges are reported to be in serious disrepair or functionally obsolete.

The use of such systems to replace existing, deteriorated bridge deck systems offers both economic benefits and improved performance. The economic advantages are possible for a number of reasons: since such composite systems are lighter, considerable savings are realized
by reduced transportation costs (several deck systems can be transported on one truck); erection costs will be less as relatively light cranes can be used to install the decks; and construction time is reduced, which eliminates long traffic delays. Due to the high resistance of FRP deck systems to environmental effects and corrosion attack, the long term performance is also expected to be improved significantly, leading to lower maintenance and longer service life. In addition to economic advantages, FRP deck systems offer structural advantages as well. For example, higher live loads can be resisted by supporting steel stringers, as the dead load applied by the FRP deck system is about one fifth of a conventional reinforced-concrete deck.

2.2 Introduction

The proposed literature review refers to different topics related to the concrete bridge decks using FRP reinforcement:

- Non-prestressed FRP reinforced bridge decks.
- Prestressed FRP reinforced bridge decks.
- Research works on shear behavior of prestressed FRP.

In this section, some of the main existing internal FRP bridge decks have been presented, so that they can be compared with the new deck system that is the subject of this thesis.

Other existing samples are in Appendix A.
2.3 Related Literature Review

2.3.1 Non-Prestressed FRP Reinforced Bridge Decks

2.3.1.1 Bridge Street Bridge, Michigan

The Bridge Street Bridge in Southfield, Michigan, is the first vehicular concrete bridge ever built in the United States that uses CFRP material as the principal structural reinforcement. The project consists of two parallel bridges - Structures A and B – over the Rouge River in the City of Southfield. Both structures use three skewed spans, each over 62 m (204 ft) long, to carry vehicular traffic. Structure A consists of a new substructure as well as a new superstructure, and incorporates five equally spaced conventional AASHTO Type III girders in each of its three spans. Its cast-in-place concrete deck slab is placed continuously across the three spans. Structure B consists of four special precast, prestressed double-tee (DT) girders in each of the three spans configured as simply supported spans. Each DT girder is structurally reinforced polymer (CFRP) Leadline™ tendons and post-tensioned carbon fiber composite cable (CFCC)™ strands in both longitudinal and transverse directions. The non-prestressed reinforcement in the girders and deck structure consists of CFCC strands manufactured in bent configurations, straight CFCC reinforcing bars, CFRP NEFMAC™ grid reinforcement, and stainless steel reinforcing bars for stirrups.

The bridge cross section (see Fig. 3) consists of four precast DT sections and a minimum 75 mm (3 in.) thick non-continuous deck slab.
The composite topping is reinforced with NEFMAC grids. The composite section is also reinforced with four externally draped 40 mm (1.57 in.) diameter unbonded CFCC post-tensioning strands. The unique construction method was based on a process developed and tested previously at LTU. The transportation of the 12 girders from the precast plant in Windsor, Ontario, to the bridge site in Southfield, Michigan, required a special barging arrangement. The girders were erected using two large capacity cranes at opposite ends of the bridge.

Fig. 4 shows the erection of a DT girder for the south span, and Fig. 5 shows the installed girders with external post-tensioning strands in place.
The Bridge Street Bridge Deployment Project has served as an extraordinarily successful example of technology transfer from research and development to serviceable structure. The bridge exhibits innovation not only in the material itself, but also in the variety of prestressing methods implemented – pretensioning and post-tensioning, internal and external. This project recently won PCI’s Harry H. Edwards Industry Advancement Award in the recent PCI Design Awards Program (see Fig. 6).
Fig. 6 - Structure B Completed and Ready for Traffic
2.3.2 Prestressed FRP Reinforced Bridge Decks

2.3.2.1 Evaluation of Frp Prestressed Panels/Slabs for I-225/Parker Road Project

Under the Innovative Bridge Research and Construction (IBRC) program of the Federal Highway Administration (FHWA), in 2001 the Colorado Department of Transportation (CDOT) has introduced fiber reinforced polymeric (FRP) reinforcement in a bridge project at I-225/Parker Road. Precast prestressed concrete panels were used as stay-in-place forms for a bridge deck. The bridge during construction is shown in Fig. 7.

![Fig. 7 - I-225/Parker Road Bridge under Construction](image)

Some of these panels were prestressed with CFRP tendons and the rest with regular seven-wire steel strands. The primary objective of this project was to demonstrate the feasibility of using CFRP tendons in place of seven wire steel strand for prestressed concrete panels.
It was the first time CFRP bars were used in such fashion. The precast panels were supported on two cast-in-place, post-tensioned concrete box girders (Fig. 8 and Fig. 9). A topping slab was added to the panels to form a composite bridge deck to carry the traffic.

![Stay-in-Place Precast Panel Forms](image1)

**Fig. 8 - Installation of Precast Panels**

![Precast Panels Supported on Two Box Girders](image2)

**Fig. 9 - Precast Panels Supported on Two Box Girders**

Furthermore, the applicability of current AASHTO provisions to CFRP prestressed panels was investigated. In addition to the experimental investigation, a new rational design methodology for
bridge decks was proposed and investigated (Borlin 2001) in this project. Most highway bridges in the United States have slab-on-girder decks, in which steel or precast concrete girders support a reinforced concrete slab. The two components are tied together with shear connectors to allow for composite action. The main reinforcement in the deck slabs is placed perpendicular to the direction of traffic. For these decks, both the AASHTO Standard Specifications for Highway Bridges (AASHTO 1996) and the conventional method in the AASHTO LRFD Bridge Design Specifications (AASHTO 1998), resulted in two dense reinforcement mats, one in the top of the slab and one in the bottom (Fig. 10 and Fig. 11).

Fig. 10 - Top Reinforcement
Pullout tests conducted in this study showed that Leadline CFRP prestressing tendons and C-BAR glass fiber reinforced polymeric (GFRP) reinforcing bars had higher bond strengths than seven-wire steel strand and regular steel reinforcing bars, respectively.

Load tests were first performed on two panels, one prestressed and reinforced with FRP and the other prestressed and reinforced with steel. Both panels were designed to barely satisfy the AASHTO specifications. An additional panel that was prestressed and reinforced with FRP and brought from the I-225/Parker Road project was tested as well. This panel was conservatively designed with a significant reserve capacity compared to the first two. Load tests were also performed on steel and FRP prestressed panels that had a 125 mm (5 in.) composite topping slab (see Fig. 12 and Fig. 13). All test results showed the feasibility of using CFRP tendons for prestressing and GFRP bars for temperature and distribution reinforcement in precast bridge panel construction. The GFRP reinforcement was selected...
according to the recommendation for temperature reinforcement in the ACI 440H draft report.

Load distribution data were taken during the composite slab tests to validate the adequacy of the Equivalent Width Strip method used in AASHTO LRFD Specifications. The method was found to be conservative for both steel and FRP reinforced composite slabs. However, results showed that the steel reinforced slab better distributed the loading in the transverse direction than the FRP reinforced slab. This suggests that the recommendation for temperature reinforcement in the ACI 440H draft report may not be adequate for distribution reinforcement.

Fig. 12 - Panel and Anchorage Details
Fig. 13 - Construction of the Panels
2.3.3 Research works on Shear Behavior of Prestressed FRP

The majority of research on concrete structures using FRP reinforcement has been on members that are not critical in shear tests. At present, the shear behaviour of prestressed concrete members using FRP reinforcement is not well understood. Unlike flexural behavior, shear behaviour is quite complex itself, even in ordinary reinforced or prestressed concrete members.

Structures are usually conservatively designed, and rely on plasticity theory for safety. This ensures that, if a set of internal forces exists which is in equilibrium with the applied load, and since it is known that steel is ductile, the lower bound (or ‘Safe Load’) theorem can be used to assert that the structure is safe.

When advanced composites are used, however, the theoretical justification is much less sound; many of the basic assumptions no longer hold. Composites are generally less stiff than steel so, when the concrete cracks, a composite is carrying less force than steel would be. Cracks will, thus, be wider, so there will be less concrete–concrete interaction across the crack; there will thus be less ‘aggregate interlock’. Composites also delaminate when placed across shear cracks, so ‘dowel action’ will be lower and there are problems caused by the bends in the bars.

Finally, and most importantly, although composites have high strain capacities, they do not behave plasticly, so the Safe Load theorem cannot be used to hide the lack of knowledge about the deflections. Taken together, these results mean that care must be taken about producing design guidelines for shear in compositely reinforced structures.
There is clearly much work to be done in this field—a model is required which satisfies all three of the basic principles of structural mechanics—equilibrium, compatibility and the material stress–strain behaviour [C.J. Burgoyne, August 2001].

Here a summary of the main research works on this topic is given.

2.3.3.1 S.Y. Park and A.E. Naaman (1999)

The authors conducted an experimental program on two series of tests (two sets of beams). The first series comprised nine prestressed concrete beams fabricated without stirrups. Five beams were prestressed using CFRP tendons and, for comparison, four beams were prestressed using conventional steel tendons.

The main objective of this first series of tests was to experimentally confirm the shear-tendon rupture failure mode in prestressed concrete beams with FRP tendons and to compare it with other failure modes in prestressed concrete beams with steel tendons.

The second series of the experimental program comprised seven FRP prestressed concrete beams and one non-prestressed concrete beam shear reinforced with steel stirrups (seven beams) or steel fibers (one beam). The test parameters were the pretensioning ratio, the shear span-to-depth ratio, shear reinforcement ratio, the use of steel fibers, the compressive strength of concrete, and the type of reinforcement.

The main goal of the second series was to evaluate the parameters affecting the shear strength and ductility of concrete beams prestressed with FRP tendons.

On the basis of this experimental investigation, the following conclusions were drawn:
1. The shear-tendon rupture failure is a unique mode of failure, which, unless properly designed for, is likely to occur in concrete beams prestressed with FRP tendons. This premature failure is due to tendon rupture by dowel shear at the shear-cracking plane. It is attributed to the poor resistance of FRP tendons in the transverse direction and their brittle behavior.

2. The ultimate shear resisting capacity of beams prestressed with FRP tendons was about 15 percent less than that of beams prestressed with steel tendons, regardless of their shear failure mode.

3. The shear-tendon rupture failure occurred at the flexural-shear-cracking plane in beams with FRP tendons, even when the effective prestress ratio was low (about 40 %) and the required amount of steel stirrups was provided according to the ACI Code.

4. Adding steel fibers is a possible way to improve the shear resistance of concrete beams prestressed with FRP tendons by avoiding or delaying shear-tendon rupture failure.

5. Differences in the properties of FRP and steel tendons appear to have no significant effect on the initial portion of load-deflection response of prestressed concrete beams subjected to a center point loading with a shear span-to-depth ratio of 2.5.

6. The ultimate shear displacement and crack width of prestressed beams that failed by shear-tendon rupture were about one-third and one-half, respectively, of those of similar beams with steel tendons.

7. The following observations were made for beams prestressed with FRP tendons:
• Increasing the shear span-to-dept ratio from 1.5 to 3.5 led to a decrease in shear resistance but an increase in shear ductility (displacement).

• Adding stirrups in sufficient quantity changes the failure mode from shear-tension to shear-tendon rupture in beams with a low effective prestress ratio of about 40 percent.

• Increasing the compressive strength of concrete slightly increases the shear strength and considerably increases the corresponding deflection.

2.3.3.2 P.A. Whitehead and T.J. Ibell (Jan. 2004)
The authors developed a model incorporating force equilibrium and compatibility of strains so that the elastic properties of FRP could be included rationally.

Fifteen specimens from the experimental program and four case-study specimens (all of which failed in shear) were used to assess the accuracy of predictions from the most prevalent codes and design guides currently in use in the UK and the US, namely BS8110 (1997), BD44 (1995), EC2 (1992) and ACI-440.1R-03 (2003). ACI-440.1R-03 is specifically intended for FRP-reinforced concrete. However, predictions obtained using modified versions of the other three standard codes were also provided. These modifications were termed the Strain Approach and the Sheffield Approach (Guadagnini et al., 2001).

The following conclusions were drawn:
1. The analytical shear predictions developed by the authors predicted the experimental results with a better accuracy when compared to the existing building codes.
2. Using either the Strain or Sheffield modification suggestions in tandem with BS8110, BD44 and EC2 results in reasonable predictions for all the reinforced (AFRP and GFRP) specimen capacities. ACI-440.1R-03 provides more conservative predictions for the FRP-reinforced specimens.

3. The presence of prestress was found to be significant in increasing shear capacity of such specimens, due to the significant crack-closing influence of the prestress. The unfactored code evaluations are more conservative for the prestressed specimens, implying that the presence of prestress aids the FRP-reinforced concrete beams to a greater extent than it does steel-reinforced concrete.

2.3.3.3 P. A. Whitehead and T. J. Ibell (Feb. 2004)

A novel FRP shear reinforcement strategy, in which both the concrete and FRP are employed to maximum advantage was conceived by the authors. They presented the findings of research conducted into the shear behaviour of FRP-reinforced and prestressed concrete beams containing continuous FRP helical transverse reinforcement. Twelve tests were conducted on ordinarily-reinforced beams and fifteen on FRP-prestressed concrete beams.

Tests on FRP-reinforced concrete beams were indeed conducted initially, thereby allowing rapid assessment of the influence of various shear reinforcement strategies which could later be investigated under prestressed conditions. Accordingly, the more effective forms of shear reinforcement were taken forward and re-examined under prestressed conditions while the non-responsive forms were discarded. Further, in
order to make direct comparison between FRP-reinforced and -prestressed beams, the effective depth was kept the same for both sets of tests.

Ibell and Burgoyne suggested that geometry and bond (including the locality of bond) are of paramount importance in the performance of FRP shear reinforcement. Therefore, to examine these aspects, the following FRP shear reinforcement strategies were employed within the FRP-reinforced and -prestressed rectangular concrete beams:

1. Circular helix, fully-bonded or entirely unbonded, placed along the top of the beam within the flexural compression zone. See Fig. 14-a.

2. Circular helix, fully-bonded or entirely unbonded, angled in the shear zones, following the lines of principal compression. See Fig. 14-b.

3. Continuous rectangular draped helix, fully-bonded, intermittently-bonded or entirely unbonded, placed over the full depth of the section. See Fig. 14-c.

4. Two interlocking rectangular draped helices, entirely unbonded, placed over the full depth of the section. See Fig. 14-d.
The following conclusions were made on shear behavior:

- The presence of prestress was significant in increasing shear capacity of such specimens.

- When used to resist shear, fully-unbonded circular and rectangular helices had to be spaced at a closer pitch in comparison with fully-bonded or intermittently-bonded rectangular helices to provide a similar increase in failure capacity. The fully-unbonded rectangular helices appeared to
have been about 50% as effective as fully- or intermittently-bonded rectangular helices in resisting shear.

- Of the arrangements considered, the best technically seemed to involve the use of a fully-bonded circular helix and a fully-bonded rectangular helix in the constant-moment region, coupled with an intermittently-bonded rectangular helix in the shear zones. This configuration led to considerable deformability and ductility and produced high shear capacity and genuine plastic-based ductility during shear collapse.

- FRP reinforcement need not simply be treated as a direct substitute for steel reinforcement, but that rather its inherent advantages should be exploited using rational design approaches.
3 EXPERIMENTAL PROGRAM

This section presents the pre-construction investigations conducted on two specimens representing a deck strip 457 mm (18 in.) wide, 254 mm (10 in.) deep and 7 m (23 ft) long, fabricated and tested. The specimens were constructed by the contractor peculiar for the project allowing for his familiarization with the use of non-conventional materials. The testing of the specimens as continuous slabs over three supports allowed the validation of the design calculations in terms of flexure and shear capacities.

3.1 Specimens Layout

Two specimens having the same geometry and amount of reinforcement were built and tested, one to investigate the flexural behaviour, the other one the shear behaviour. The specimens were reinforced using 3 $f_{19}$ (6/8 in) GFPR bars as top and bottom mat and 2 $f_{9}$ (3/8 in) CFRP bars as prestressed tendons. The position of the prestressed tendons was varied along the slab in order to match the moment demand. In addition, in order to reproduce the actual field conditions also $f_{13}$ (4/8 in) GFRP bars spaced 305 mm (12 in) on center were placed in the transversal direction as temperature and shrinkage reinforcement. Fig. 15 shows a detailed layout of the reinforcement, while Fig. 16 shows the position of the CFRP tendons.
3.2 Material Properties

Tests were performed to characterize the mechanical properties of the materials used in this investigation.

The designed concrete compressive strength was equal to 41.4 MPa (6000 psi). Water to cement ratio for the concrete mixture was 0.45. The components in the concrete mixture were proportioned as follow by weight: 19% portland cement, 40% crushed limestone, 33% sand, and 8% water.
The actual compressive strength of the concrete was checked on three 100 x 200 mm (4 x 8 in) cylinders per slab. The cylinders were tested in compliance with ASTM C39/C39M. The average compressive strength at the time of the test was found equal to 48.6 MPa (7040 psi) and 45.4 MPa (6585 psi) for the flexural and shear specimens, respectively.

The compressive strength of the high performance cementitious grout was determined on three cylinders 100 x 200 mm (4 x 8 in) and it was found to be 21.7 MPa (3150 psi) after one day, 36.5 MPa (5300 psi) after three days, 49.3 MPa (7150 psi) after seven days, and 58.9 MPa (8550 psi) after 28 days. In addition, splitting tensile tests in compliance with ASTM C496 were performed on the same type of cylinders (three repetitions). The splitting tensile strength was found to be 1.5 MPa (218 psi) after one day, 2.8 MPa (406 psi) after three days, 3.58 MPa (520 psi) after 7 days, and 5.59 MPa (810 psi) after 28 days.

Tensile tests were performed on FRP bars to determine their mechanical properties which are related to fiber content. The average tensile strength, ultimate strain and modulus of elasticity obtained from the testing of the specimens (ASTM D3039) are presented in Table 1.
Table 1 - Mechanical Properties of FRP Bars

<table>
<thead>
<tr>
<th>Bar Type</th>
<th>Nominal Diameter of the Bar mm (in)</th>
<th>Average Max. Strain %</th>
<th>Average Max. Stress MPa (ksi)</th>
<th>Average Elastic Modulus GPa (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GFRP Bar</td>
<td>12.7 (0.5)</td>
<td>1.68</td>
<td>689.5 (100.0)</td>
<td>40.80 (5920)</td>
</tr>
<tr>
<td>GFRP Bar</td>
<td>19.1 (0.75)</td>
<td>1.52</td>
<td>620.5 (90.0)</td>
<td>40.80 (5920)</td>
</tr>
<tr>
<td>CFRP Bar</td>
<td>9.5 (0.375)</td>
<td>1.67</td>
<td>2124.2 (308.1)</td>
<td>142.7 (20702)</td>
</tr>
</tbody>
</table>

3.3 Specimens Design

The theoretical moment and shear capacities have been computed according to ACI 440.1R-03 provisions. As an alternative method to compute the shear capacity of the specimens the equation developed by Tureyen A. K. and Frosh R. J. and now under consideration for adoption by ACI Committee 440, was used.

According to the ACI 440.1R-03 the nominal flexural capacity of an FRP reinforced concrete member can be computed as follows:

$$M_n = A_f f_{fu} (d - \frac{\beta_i c}{2}) + A_{fcp} f_{cp} (d_p - \frac{\beta_i c}{2}) \quad (3.1)$$

where:

- $A_f = \text{area of FRP reinforcement}$
- $f_{fu} = \text{design tensile strength of FRP, considering reductions for service environment}$
- $d = \text{distance from extreme compression fiber to centroid of tension reinforcement}$
- $\beta_i = \text{factor depending on the concrete strength, } f'_c, \text{ equal to 0.75 for } f'_c = 41.4 \text{ MPa (6000 psi)}$
\[ c = \text{distance from extreme compression fiber to the neutral axis}\]

The second term symbols represent the same factors but referred to the prestressed reinforcement.

The theoretical moment capacity is 97.9 kN-m (72.2 kip-ft), while without the post-tension reinforcement this value becomes 76.0 kN-m (55 kip-ft).

According to the ACI 440.1R-03 the concrete shear capacity \( V_{c,f} \) of flexural members using FRP as main reinforcement can be evaluated as shown below. The proposed equation accounts for the axial stiffness of the FRP reinforcement \( (A_fE_f) \) as compared to that of the steel reinforcement \( (A_sE_s) \).

\[
V_{c,f} = \frac{A_f E_f}{A_s E_s} V_c \quad \text{(3.2)}
\]

\( V_c \) being the nominal shear strength provided by concrete with steel flexural reinforcement, for members with effective prestress force not less than 40 percent of the tensile strength of flexural reinforcement, according to ACI 318R-99:

\[
V_c = (0.6700) \left[ V_u d_p \right] + \left[ 700 \frac{V_u d_p}{M_u} bd_p \right] \quad \text{(3.3)}
\]

where:

\( f_c' = \text{specified compressive strength of concrete} \)

\( V_u = \text{factored shear force at section} \)

\( d_p = \text{distance from extreme compression fiber to centroid of prestressed reinforcement} \) (but needs not to be less than 0.80h for circular sections and prestressed members)

\( M_u = \text{factored moment at section} \)
When assuming the $C_E$ (environmental reduction factor) equal to 1, the theoretical shear capacity will be 69.8 kN (15.7 kip). According to Tureyen A. K. and Frosh R. J. the shear capacity can be derived from the following equation:

$$V_{c,t} = 5\sqrt{f_c}bc$$  \hfill (3.4)

where $c$ is the position of the neutral axis at the service conditions, $b$ is the width of the specimens and $f_c$ the compressive strength of the concrete. The shear capacity computed according to this approach is 97.4 kN (21.9 kip).

Indeed using the neutral axis at the service conditions may not be correct when the member is approaching its flexural capacity; hence, assuming $c$ corresponding to the ultimate conditions of the section, the shear capacity will be 54.7 kN (12.3 kip).

The prestressing was primarily used to increase the shear capacity of the slabs rather than the flexural one. In fact, the shear capacity of the slabs without any prestressing load would have been 30.7 kN (6.9 kip) and 33.8 kN (7.6 kip) using the ACI and the Tureyen A. K. and Frosh R. J., respectively, versus 69.8 kN and 97.4 kN with post-tensioned reinforcement, respectively. Hence, according to ACI, the post-tension led to an increase in flexure of ~29%, while the shear is more than doubled.
3.4 Specimens Preparation

The specimens were built in the way to reproduce the same characteristics of the bridge deck that would be constructed in the field.

The CFRP tendons were housed inside a plastic duct, in order to allow the post-tensioning operations. Plastic T joints were used to connect the duct housing the tendons with the duct going out of the specimen to inject the grout.

Plastic chairs and ties were used to lay the bars and the tendons in order to have a completely “steel free” structure, in compliance with the requirements of the Southview bridge project. Fig. 17 shows the cage details of both specimens.

![Fig. 17 - Cage Detail](image)

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 (see Fig. 18) and on the compressive face of the slab. Furthermore Fig. 19 shows the two specimens ready for pouring.
The prestressing of the tendons was executed 28 days after the pouring of the concrete. Fig. 20 and Fig. 21 show the specimens while curing and after the removal of the formwork.
The CFRP bars were prestressed by applying a force of 98 kN (22 kip) using hydraulic jacks at both ends. This level of prestressing 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 (supposed to be 35% of the initial strain of prestress reinforcement), the creep rupture limits dictated for GFRP bars according to ACI 440.1R-03 (0.20 times the ultimate guaranteed
tensile strength for prestress CFRP reinforcement), as underlined in the next section.
Initially the prestressing load was applied only to 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 prestressing load in steps of 31 kN (7 kip) from both ends by mean of two hydraulic jacks (See Fig. 22). The prestressing load was monitored using two load cells, one for each end of the slab, while the strain was measured 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 Fig. 23). The inner sleeve was made out of copper/steel and it was 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.

The prestressing of the CFRP bars was followed by their grouting using a high performance cementitious grout. The cementitious grout was allowed to cure for seven days after which the anchoring of the tendons was removed by drilling the CFRP bar inside the barrel (see Fig. 24). The strain in the slab was monitored during and for 48 hours after the cutting of the anchoring system: during this time no loss of compressive strain in the specimens was recorded.
3.5 Test Setup

Each slab was tested as a continuous member on three supports, comprising of a 3.6 m (12 ft) and 1.8 m (6 ft) spans. 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-span for the flexural specimen:

Given:

\[
\begin{align*}
L_1 &= 3.6 \text{m (12 ft)}  \\
L_2 &= 1.8 \text{m (6 ft)}  \\
P_1 &= 165kN (37kip)  \\
P_2 &= 100kN (23kip)
\end{align*}
\]

First live load

Second live load (higher than the theoretic value)

Solving the hyperstatic scheme (see Fig. 26) it’s possible to derive the flexural moment acting on the central support (assigned as hyperstatic unknown):
Fig. 26 - Hyperstatic Scheme

\[ \phi_B^L = \phi_B^R \]

\[ M = -\frac{3}{16} \left( P_1 \cdot L_1^2 + P_2 \cdot L_2^2 \right) \frac{L_1 + L_2}{L_1} \]  \hspace{1cm} (3.5)

Hence the reactions of each support can be derived:

\[ R_1 = \frac{P_1}{2} - \frac{M}{L_1} = 13.2 \text{kip} \]  \hspace{1cm} (3.6)

\[ R_2 = \frac{P_2}{2} + \frac{M}{L_1} + \frac{P_1}{2} + \frac{M}{L_2} = 46 \text{kip} \]  \hspace{1cm} (3.7)

\[ R_3 = \frac{P_2}{2} - \frac{M}{L_2} = 0.8 \text{kip} \]  \hspace{1cm} (3.8)

Finally the equation of flexural moment and its plot (see Fig. 27) depending on z can be derived:

\[ M_1(z) = R_1 \cdot z \]  \hspace{1cm} when \hspace{1cm} \[ z \in \left[ 0, \frac{L_1}{2} \right] \]

\[ M_2(z) = M_1(z) - P_1 \cdot (z - x) \]  \hspace{1cm} when \hspace{1cm} \[ z \in \left[ \frac{L_1}{2}, L_1 \right] \]

\[ M_3(z) = M_2(z) + R_2 \cdot (z - L_1) \]  \hspace{1cm} when \hspace{1cm} \[ z \in \left[ L_1, L_1 + \frac{L_2}{2} \right] \]

\[ M_4(z) = M_3(z) - P_2 \cdot [z - (L_1 - y)] \]  \hspace{1cm} when \hspace{1cm} \[ z \in \left[ L_1 + \frac{L_2}{2}, L_2 \right] \]
Fig. 27 - Flexural Plot

\[ M_{\text{max}}^-(z) = -86.9kN\cdot m(-64.1kip\cdot ft) \]
\[ M_{\text{max}}^+(z) = 107kN\cdot m(78.9kip\cdot ft) \]

Regarding the shear test, the loading points were placed 0.9 m (3 ft) away from the central support and the distance was determined by performing the following calculations (See Fig. 28):

Fig. 28 - Static Scheme for the Shear Specimen

According to the same procedure used to solve the flexural scheme, and given:
The shear equations depending on z and the corresponding plot (see Fig. 29) where derived:

\[ V_1 = R_1 \quad (3.13) \quad \text{when} \quad z \in [0, 2.7m] \]

\[ V_2 = V_1 - P_1 \quad (3.14) \quad \text{when} \quad z \in [2.7m, L_1] \]

\[ V_3 = V_2 + R_2 \quad (3.15) \quad \text{when} \quad z \in [L_1, L_1 + 0.9m] \]

\[ V_4 = V_3 - P_2 \quad (3.16) \quad \text{when} \quad z \in [L_1 + 0.9m, L_2] \]

\[ V_{max}^+ (z) = -198kN (-44.5\text{kip}) \]

\[ V_{max}^- (z) = 173.5kN (39\text{kip}) \]

The load was applied in cycles of loading and unloading. An initial cycle for a low load was performed on each specimen to verify that both the mechanical and electronic equipment was working properly.
Loads were applied to 102 x 457 x 25 mm (4 x 18 x 1 in) steel plates resisting on the slab in order to prevent that the structure could touch the edge of the support in the case of large deflections (see Fig. 30). The flexural specimen (See Fig. 31 –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 actual distribution of moments in the slab.

The loads were applied by means of 30 ton (66 kip) hydraulic jacks reacting against a steel frame. The loading rate was the same for the two spans until reaching 85 kN (19 kip). After that, the load in the shorter span was kept constant while the one in the longer one was increased up to 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. The strain gages were placed on each GFRP bar at the location of the loading points and of 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.
The loads were applied to 102 x 457 x 25 mm (4 x 18 x 1 in) steel
plates using a 100 ton (220 kip) hydraulic jack (see Fig. 31 –b). Two
additional LVDTs were inserted in order to measure also the
maximum displacement which, in this case, was not at the loading
points.

**Fig. 31 - Test Setup (all dimensions in mm)**
3.6  Test Results and Discussion

3.6.1 Failure Modes

For the “flexural” specimen, the first flexural crack was observed on the longer span when the load was approximately 66 kN (15 kip) on both spans. As the loads were increased, some of the cracks started to extend diagonally to form shear cracks (see Fig. 32).

![Shear Crack](Fig. 32 - Shear Cracks on the Flexure Specimen)

The maximum forces, 163 kN (37 kip) and 100 kN (23 kip) on long and short spans, respectively, represented the maximum load determining concrete crushing on the top of the longer span, indicating brittle flexural failure (see Fig. 33).
This was immediately followed by a sudden shear failure which also caused the rupture of the CFRP bars due to kinking (see Fig. 34).

About the “shear” specimen, the first crack was observed at a load approximately of 89 kN (20 kip) in correspondence of the central support where the moment was maximum (see Fig. 35). 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 (61 kip) due to diagonal tension shear.

Fig. 35 - Failure in the Shear Specimen

### 3.6.2 Discussion of Test Results

Table 2 compares the experimental results with the theoretical moment and shear capacities of the slabs computed according to ACI 440 provisions and to the equation developed by Tureyen A. K. and Frosh R. J.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Experimental</th>
<th>Theoretical</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maximum Bending Moment kN-m (kip-ft)</td>
<td>Maximum Shear Force kN (kip)</td>
</tr>
<tr>
<td>Flexure</td>
<td>101.87 (73.7)</td>
<td>99.7 (22.4)</td>
</tr>
<tr>
<td>Shear</td>
<td>70.2 (51.8)</td>
<td>142.3 (32.0)</td>
</tr>
</tbody>
</table>

$^{(1)}$ Computed according to ACI 440 provisions  
$^{(2)}$ Computed according to Tureyen A. K. and Frosh R. J.  
$^{(3)}$ Computed with a modified Tureyen A. K. and Frosh R. J. considering c at ultimate

From Table 2 it can be observed that for the flexural specimen the experimental shear results largely overcame the theoretical shear capacities of the specimen when calculated according to ACI 440. For the same specimen the Tureyen A. K. and Frosh R. J. approach
overestimated the shear capacity when using the neutral axis at the service conditions. By removing such assumption and taking c corresponding to the ultimate conditions of the section, a more conservative approach can be attained as shown in the last column of Table 2.

The shear specimen presented a shear capacity larger than the theoretical, demonstrating again the safe approach of ACI 440. For this specimen the maximum theoretical bending moment determined from the Tureyen A. K. and Frosh R. J. approach was only half of the ultimate moment capacity experimented in the test.

Fig. 36 shows the different trend of measured moment compared to the theoretical one derived solving the hyperstatic sheme (worked out in the Test Setup), both related to the central support reaction, $R_2$.

![Fig. 36 - M_{measured} Vs. M_{theoretical}](image-url)
Fig. 37 shows the load deflection diagram. The change of slope in the diagram corresponds to the cracking of the specimen.

![Load deflection diagram](image)

**Fig. 37 – Load deflection Diagram for the Longer Span of the Flexural Specimen**

The same change can be observed, for the same specimen at the same location, in the diagram of Fig. 38, which shows a comparison between the experimental and the theoretical moment curvature diagrams for the “flexure” specimen. The experimental diagram was determined on the longer span in correspondence of the maximum bending moment. Compared with the experimental curve, the theoretical moment curvature diagram showed a lower slope leading to overestimate the deflection of the bridge.
Fig. 38 – Comparison Between Experimental and Theoretical Moment-Curvature Diagrams

3.7 Conclusions

On the basis of the experimental investigation, it can be concluded that GFRP bars as passive and CFRP bars as active internal reinforcement could represent a feasible solution replacing the steel reinforcement of concrete slab bridges. The prestressing material is mostly needed for shear purpose rather than for flexure. According to ACI 440, the shear capacity increases from 30.7 kN (6.9 kip) with mild reinforcement to 69.8 kN (15.7 kip) adding the post-tensioned reinforcement, while the flexural capacity increases from 76.0 kN-m (55 kip-ft) to 97.9 kN-m (72.2 kip-ft). Finally, the tools provided by ACI 440 provide a safe design at both service and ultimate conditions.
4 SOUTHVIEW BRIDGE DESIGN

4.1 Introduction

In the following section, the analytical procedures used in the widening of the Southview Bridge, located in Rolla, Phelps County, MO, are summarized. The expansion phase included the removal of the existing curb from the existing reinforced concrete deck to allow the construction of two new structures adjacent to the original deck in order to extend the width of the bridge from 3.9 m (13 ft) to 11.9 m (39 ft). The curb-to-curb width of the resulting bridge is 9.1 m (30 ft). The two new structures consist of a FRP prestressed/reinforced concrete deck and a steel reinforced concrete deck as shown in Fig. 39.

![Fig. 39 - Bridge T0530](image)

The new structure is a box culvert. It consists of a steel reinforced concrete slab about 25.4 cm (10 in.) thick, as depicted in Fig. 40. The slab deck is continuous over three intermediate reinforced concrete vertical walls, and the overall length of the bridge is roughly 12 m (40 ft). The new deck was built on conventional reinforced concrete (RC) walls. The number of walls is identical to the existing number.
The objective of this section is to provide the structural analysis of the new FRP concrete bridge deck based on the AASHTO HS20-44 design truck (a detailed analysis is presented in APPENDIX B), and provide calculations for its design using a combination of non-traditional corrosion-resistant composites materials.

4.2 Assumptions

The following assumptions are made:

a) Nominal properties for FRP reinforcing material are 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 (ACI 440 in the followings);

b) Load configurations are consistent with AASHTO Specifications;

c) Design carried out according to ACI 440; and

d) Effects due to the skew are neglected.
4.3 Structural Analysis

4.3.1 Load Combinations

For the structural analysis of the bridge the definitions of the design truck and design lane are necessary. This will be addressed in the next paragraph.

Ultimate values of bending moment and shear force are obtained (in the following Summary of the Analysis) by multiplying their nominal values by the dead and live load factors and by the impact factor according to AASHTO Specifications as shown in Eq. (4.1):

\[ \omega_u = 1.3[\beta_d D + 1.67(L + I)] \]  

(4.1)

where \( D \) is the dead load, \( L \) is the live load, \( \beta_d = 1.0 \) as per AASHTO Table 3.22.1A, and \( I \) is the live load impact calculated as follows:

\[ I = \frac{50}{L+125} = \frac{50}{10+125} = 0.37 > 0.30 \]  

(4.2)

and \( L = 3.0 \text{ m (10 ft)} \) represents the span length from center to center of support. The impact factor should not be larger than 0.30, and therefore the latter value is assumed for the design.

4.3.2 Design Truck and Design Lanes

The analysis of the bridge is carried out for an HS20-44 design truck load having geometrical characteristics and weight properties shown in Fig. 41.
Two loading conditions are required to be checked as depicted in Fig. 42. The HS20-44 design truck load (Fig. 42-a) has a front axle load of 35.6 kN (8.0 kips), second axle load, located 4.3 m (14.0 ft) behind the drive axle, of 142.3 kN (32.0 kips), and rear axle load also of 142.3 kN (32.0 kips). The rear axle load is positioned at a variable distance, ranging between 4.3 m (14.0 ft) and 9.1 m (30.0 ft). Given the specific bridge geometry, the worst loading scenario is obtained for the minimum spacing of 4.3 m (14.0 ft) between the two rear axles. The design lane loading condition consists of a load of 9.3 kN/m (640 lbs/ft), uniformly distributed in the longitudinal direction with concentrated loads so placed on the span as to produce maximum stress. The concentrated load and uniform load are considered to be applied over a 3.0 m (10ft) width on a line normal to the center line of the lane. The intensity of the concentrated load is represented in Fig. 42-b for both bending moments and shear forces. This load shall be placed in such positions within the design lane as to produce the maximum stress in the member.
4.3.3 Deck Analysis

The deck slab is considered to be a one-way slab system. The width of the slab, \( E \), to be used in the analysis is provided by AASHTO (Section 3.24.3.2) as follows:

\[
E = 4 + 0.06S = 4 + 0.06(10 \text{ ft}) = 1.4m (= 4.6 \text{ ft}) \quad (4.2)
\]

where \( S \) represents the slab length assumed equal to \( 3.0 \text{ m} \) (\( 10 \text{ ft} \)).

4.3.4 Flexural and Shear Analysis

Fig. 43 shows a lateral view of the bridge deck when an HS20-44 design truck moves from the right to the left as the value of \( x_1 \) increases from \( 0 \) to \( L \), where \( L \) represents the total bridge length.
The values of $P_i$ ($i=a,b,c$) represent the wheel load as defined by AASHTO (~18, 71 and 71 kN, namely 4, 16 and 16 kips, respectively).

Table 3 summarizes values reported in Fig. 43 and reports parameters used in the calculation of the moment and shear due to dead load. The analysis and design are carried out for a unit-width strip (~30 cm, 12 in) of slab deck.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Span Length, $L_1$</td>
<td>2.89 m (9.49 ft)</td>
</tr>
<tr>
<td>Second Span Length, $L_2$</td>
<td>3.38 m (11.10 ft)</td>
</tr>
<tr>
<td>Third Span Length, $L_3$</td>
<td>3.30 m (10.82 ft)</td>
</tr>
<tr>
<td>Fourth Span Length, $L_4$</td>
<td>2.85 m (9.35 ft)</td>
</tr>
<tr>
<td>First Load, $P_a$</td>
<td>17.8 kN (4 kip)</td>
</tr>
<tr>
<td>Second Load, $P_b$</td>
<td>71.2 kN (16 kip)</td>
</tr>
<tr>
<td>Third Load, $P_c$</td>
<td>71.2 kN (16 kip)</td>
</tr>
<tr>
<td>Concrete Unit Weight, $\gamma_c$</td>
<td>24 kN/m$^3$ (150 p/ft$^3$)</td>
</tr>
<tr>
<td>Asphalt Unit Weight, $\gamma_a$</td>
<td>17.3 kN/m$^3$ (108 p/ft$^3$)</td>
</tr>
<tr>
<td>Slab Unit-Width, $b$</td>
<td>30.5 cm (12 in)</td>
</tr>
<tr>
<td>Slab Height, $h$</td>
<td>25.4 cm (10 in)</td>
</tr>
<tr>
<td>Asphalt Thickness, $t$</td>
<td>15.2 cm (6 in)</td>
</tr>
</tbody>
</table>

As the design truck moves from the right to the left side of the bridge, fourteen different loading conditions are defined, as shown in Fig. 44.
Three more loading conditions related to the design lane will be analyzed as reported in Fig. 45. The first loading condition is related to the maximum positive moment, the second one to the maximum negative moment, and the third one to the maximum shear.
Fig. 46 shows the moment diagram as the design truck moves on the bridge following the fourteen loading phases highlighted in Fig. 44. Fig. 47 shows the moment diagram due to the slab and asphalt layer self-load. Both diagrams are drawn for a ~30 cm (12 in) strip-width.

**Fig. 46** - Unfactored Bending Moment Diagrams Due to Live Load
Ultimate values are obtained by taking into account the maximum positive and negative moment from Fig. 46 with the load factors summarized in Eq. (4.1), and by adding the corresponding moment due to the dead load (Fig. 47) with the load factors taken from the same equation.

The same diagrams can be drawn for shear as reported in Fig. 48 and Fig. 49 for live and dead load, respectively, and for a \(~30 \text{ cm} (12 \text{ in})\) wide unit-strip.
Fig. 48 - Unfactored Shear Diagrams Due to Live Load

Fig. 49 - Unfactored Shear Diagrams Due to Dead Load

It has to be underlined that both moment and shear diagrams due to the dead load have been calculated using a simplified structure where the distances between supports were assumed to be equal to 3.0 m (10 ft).
Similarly, moment and shear diagram related to the design lane loading condition can be found as depicted in Fig. 50 for the three structures shown in Fig. 45. The design has been performed for a ~30 cm (12 in) unit strip.

**Maximum Positive Moment**

**Maximum Negative Moment**
4.3.5 Summary of the Analysis

Bending moments and shear forces are summarized in Table 4. Columns (1) and (3) represent the factored coefficient to be applied to dead and live load, respectively. Columns (2) and (4) show both unfactored dead and live load moment and shear, respectively, as taken from the previous figures.
Table 4 - 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></td>
<td>(1)</td>
<td>(2)</td>
<td>(3)</td>
<td>(4)</td>
</tr>
<tr>
<td>Design Truck</td>
<td></td>
<td></td>
<td></td>
<td>(5)=(1)(2)+(3)(4)</td>
</tr>
<tr>
<td>M^+ [kN-m/m]</td>
<td>6.27</td>
<td>29.69</td>
<td>92.09</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(1.38 k-ft/ft)</td>
<td>(6.55 k-ft/ft)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M^- [kN-m/m]</td>
<td>8.69</td>
<td>23.52</td>
<td>77.56</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(1.92 k-ft/ft)</td>
<td>(5.19 k-ft/ft)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear [kN /m]</td>
<td>4.92</td>
<td>15.42</td>
<td>160.53</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(1.09 kip/ft)</td>
<td>(3.40 kip/ft)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design Lane</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>M^+ [kN-m/m]</td>
<td>6.27</td>
<td>18.21</td>
<td>59.41</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(1.38 k-ft/ft)</td>
<td>(4.01 k-ft/ft)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M^- [kN-m/m]</td>
<td>8.69</td>
<td>17.29</td>
<td>59.88</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(1.92 k-ft/ft)</td>
<td>(3.81 k-ft/ft)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear [kN /m]</td>
<td>4.92</td>
<td>12.47</td>
<td>134.25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(1.09 kip/ft)</td>
<td>(2.75 kip/ft)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Since the design truck analysis produces the highest stresses on the slab, only moment and shear related to this analysis will be considered.

Fig. 51 shows the envelope of the bending moment diagram obtained when the truck travels on the bridge following all the loading conditions summarized in Fig. 44.

Fig. 51 - Bending Moment Envelopes (kN-m/m)
4.3.6 Deflections

The worst loading condition scenario for calculating the slab deck deflections is reported in Fig. 52. Two concentrated loads simulating the truck wheels are applied at mid-span of the second and fourth span. For this analysis, the span lengths have been assumed equal to 30 m (10 ft). To further simplify the analysis, the fourth span of Fig. 52-a could be separately analyzed as depicted in Fig. 52-b. Only the latter approach is presented here, and the obtained results need to be considered as an upper bound limit.

![Fig. 52 - Bridge Deflection Analysis (Live Load)](image)

The deflection due to the dead load (as depicted in Fig. 53) needs to be added to the live load deflections of Fig. 52.

![Fig. 53 - Bridge Deflection Analysis (Dead Load)](image)
The two settlements can be written as follows:

\[
\Delta_{LL} = \frac{8 P l^3}{384 E_s I_s} \\
\Delta_{DL} = 0.0065 \frac{w_d l^4}{E_s I_s}
\]  

(4.3)

where \( P = 1.3 \times 71.2 \text{ kN (1.3*16 kips)} \) is the wheel load increased by the impact factor, \( w_d \sim 2.6 \text{ kN/m (0.179 k/ft)} \) represents the dead load of the bridge, and \( E_s \) and \( I_s \) are the modulus of elasticity of the concrete and the moment of inertia of the cross-section of the slab, respectively. The value of the moment of inertia will change depending on whether the cross-section can be considered cracked or uncracked.

### 4.4 Design

#### 4.4.1 Assumptions

The design of the internal FRP reinforcement is carried out according to the principles of ACI 440. The properties of concrete, steel and FRP bars used in the design are summarized in Table 5. The reported FRP properties are guaranteed values.
Table 5 - Material Properties

<table>
<thead>
<tr>
<th>Concrete Compressive Strength $f_c$ (MPa)</th>
<th>FRP Internal Reinforcement Type</th>
<th>FRP Bar Size</th>
<th>FRP Tensile Strength $f_{fu}$ (MPa)</th>
<th>FRP Tensile Strain $\varepsilon_{fu}$</th>
<th>FRP Modulus of Elasticity $E_f$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>41.4 (6,000 psi)</td>
<td>GFRP</td>
<td>f 9 (#3)</td>
<td>758 (110 ksi)</td>
<td>0.018</td>
<td>40.8 (5,920 ksi)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>f 13 (#4)</td>
<td>689 (100 ksi)</td>
<td>0.017</td>
<td>40.8 (5,920 ksi)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>f 19 (#6)</td>
<td>621 (90 ksi)</td>
<td>0.015</td>
<td>40.8 (5,920 ksi)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>f 22 (#7)</td>
<td>586 (85 ksi)</td>
<td>0.014</td>
<td>40.8 (5,920 ksi)</td>
</tr>
<tr>
<td></td>
<td>CFRP</td>
<td>f 19 (#3)</td>
<td>2068 (300 ksi)</td>
<td>0.017</td>
<td>124.1 (18,000 ksi)</td>
</tr>
</tbody>
</table>

Material properties of the FRP reinforcement reported by manufacturers, such as the ultimate tensile strength, typically do not consider long-term exposure to environmental conditions, and should be considered as initial properties. FRP properties to be used in all design equations are given as follows (ACI 440):

$$ f_{fu} = C_E f_{fu}^* $$
$$ \varepsilon_{fu} = C_E \varepsilon_{fu}^* $$ (4.4)

where $f_{fu}$ and $\varepsilon_{fu}$ are the FRP design tensile strength and ultimate strain considering the environmental reduction factor ($C_E$) as given in Table 7.1 (ACI 440), and $f_{fu}^*$ and $\varepsilon_{fu}^*$ represent the FRP guaranteed tensile strength and ultimate strain as reported by the manufacturer (see Table 5). The FRP design modulus of elasticity is the average value as reported by the manufacturer.
4.4.2 Slab Design

4.4.2.1 Flexural Design

The flexural design of a FRP reinforced concrete member is similar to the design of a steel reinforced concrete member. The main difference is that both concrete crushing and FRP rupture are potential mechanisms of failure. As an FRP reinforced concrete member is usually less ductile than the correspondent steel reinforced concrete member, the strength reduction factor, $\phi$, needs to be revisited according to Eq. (4.5) (ACI 440):

$$
\phi = \begin{cases} 
0.50 & \text{if } \rho_f \leq \rho_{fb} \\
\frac{\rho_f}{2\rho_{fb}} & \text{if } \rho_{fb} < \rho_f < 1.4\rho_{fb} \\
0.70 & \text{if } \rho_f \geq 1.4\rho_{fb}
\end{cases}
$$

(4.5)

where $\rho_f$ is the FRP reinforcement ratio and $\rho_{fb}$ represents the FRP reinforcement ratio producing balanced failure condition.

Fig. 54 shows the trend of the strength reduction factor as a function of both concrete compressive strength and the number of GFRP bars used.
By increasing the concrete compressive strength, the value of the $\phi$ factor changes from 0.7 to 0.5 as the failure mode moves from concrete crushing to FRP rupture.

Fig. 55 shows the nominal and factored flexural capacities of the bridge deck as a function of the concrete compressive strength and the number of FRP bars installed (same legend of Fig. 54).

Table 6 summarizes the properties and the flexural capacity of the bridge deck corresponding to a cross section with a $f_{19}$ (#6) Aslan
100 GFRP as mild reinforcement at \(~15\ cm\ (6\ in)\) center-to-center. Prestressing FRP tendons made out of f9 (#3) Aslan 200 CFRP bars are installed at \(~23\ cm\ (9\ in)\) center-to-center and post-tensioned after the concrete deck is cured. Calculations are carried out for a \(~30\ cm\ (12\ in)\) unit strip.

<table>
<thead>
<tr>
<th>Overall Height of the Slab, (h) [cm]</th>
<th>Area of Tension GFRP Reinforcement, (A_t) [cm(^2)/m]</th>
<th>GFRP Effective Depth, (d) [cm]</th>
<th>Area of Prestressed CFRP Tendons, (A_p) [cm(^2)/m]</th>
<th>CFRP Effective Depth, (d_p) [cm]</th>
<th>Flexural Capacity (\phi M_n) [kN-m/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>25.4 (10 in)</td>
<td>19.39 (0.916 in(^2)/ft)</td>
<td>20.6 (8.125 in)</td>
<td>2.13 (0.101 in(^2)/ft)</td>
<td>17.8 (7.0 in)</td>
<td>92.8 (20.5 k-ft/ft)</td>
</tr>
</tbody>
</table>

The above flexural capacity is related to both positive and negative moment regions, and it is larger than the required demand previously shown in Table 4.

The prestress strain in the tendons is equal to 65% of their ultimate strain. All CFRP losses are assumed to be 30% of the initial prestressing strain.

4.4.2.2 Shear Design

The shear capacity of FRP reinforced concrete sections is calculated following the principles of ACI 440. Particularly, the concrete contribution to the shear capacity, \(V_{c,f}\), can be expressed as follows:

\[
V_{c,f} = A_t E_t V_c \frac{A_p E_p}{A_t E_t} \quad (4.6)
\]
where the ratio \( (A_f E_f / A_s E_s) \) takes into account the axial stiffness of the FRP reinforcement as compared to that of steel reinforcement, and \( V_c \) is given as follows (ACI 318-99):

\[
V_c = (0.6\sqrt{f_c} + 700 \frac{V_d p}{M_p}) b d \rho \quad (4.7)
\]

\( A_s \) reported in Eq. (4.6) can be found by determining the area of steel reinforcement required to match the factored FRP flexural capacity, \( \phi M_n \). Table 7 summarizes the steel and GFRP design and the assumed value of \( A_s \).

<table>
<thead>
<tr>
<th>Description</th>
<th>Steel Design</th>
<th>GFRP Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck Slab, ( h )</td>
<td>25.4 cm (10 in)</td>
<td>25.4 (10 in)</td>
</tr>
<tr>
<td>Deck Width, ( b )</td>
<td>30.5 cm (12 in)</td>
<td>30.5 (12 in)</td>
</tr>
<tr>
<td>Effective Depth, ( d )</td>
<td>20.6 cm (8.125 in)</td>
<td>20.6 (8.125 in)</td>
</tr>
<tr>
<td>Reinforcement Area</td>
<td>( f 13@25cm=11.2cm^2/m ) (#4@10&quot;=0.528 in²/ft)</td>
<td>( f 19@11cm=25.3 cm^2/m ) (#6@4.5&quot;=1.195 in²/ft)</td>
</tr>
<tr>
<td>Factored Flexural Capacity, ( \phi M_n )</td>
<td>94.3 kN-m/m (20.8 k-ft/ft)</td>
<td>93.0 kN-m/m (20.5 k-ft/ft)</td>
</tr>
</tbody>
</table>

The concrete contribution to the shear capacity of the member yields:

\[
V_c = \left(0.6\sqrt{6000} + 700 \frac{11.0(7/12)}{17.1}\right)(12)(7) = 379.4kN / m(= 26.0 \text{ kips / ft}) \quad (4.8)
\]

and from Eq. (4.6):

\[
V_{c,j} = \frac{(1.195)(5.920)}{(0.528)(29,000)} \times 26.0 = 175kN / m(= 12.0 \text{ kips / ft}) \quad (4.9)
\]

Finally, the factored shear capacity is \( \phi V_n = \phi V_{c,j} = 0.85 \times 175 = 148.9 \text{ kN/m (=10.2 kips/ft)} \). This value is slightly smaller than the shear demand \( V_n = 160.5 \text{ kN/m (=11.0 kips/ft, see Table 4)} \). The value can be
accepted because the analysis has been performed on the centerline of the support, while it is allowed to evaluate both $V_u$ and $M_u$ at a cross section flush with the vertical wall representing the support. In this case, $V_u$ will not change substantially, while $M_u$ will have an appreciable lower value. Therefore (while keeping $V_ud_p/M_u<1$), $V_c$ expressed by Eq. (4.8) will increase.

Consistent with section 3 it is to be noted that the prestressing force is most needed for shear purpose rather than flexure. In fact, if the prestressing action was not considered, the concrete contribution to the shear capacity would have been $V_c=220.4$ kN/m (=15.1 kips/ft), and the final factored shear capacity of the bridge would have been equal to $\phi V_n=86.09$ kN/m $<< 148.9$ kN/m ($\phi V_n=5.9$ kips/ft $<< 10.2$ kips/ft); therefore the post-tensioning allowed to increase the shear capacity of the slab more than 40%.

As an alternative approach to the shear capacity of the bridge deck, the following equation (Tureyen A. K. and Frosh R. J., 2003), now under consideration for adoption by ACI Committee 440, could be used:

$$V_{c,f} = 5\sqrt{f'_c} bc$$  \hspace{1cm} (4.10)

where $c$ is the position of the neutral axis at service. This approach is justified by the parametric analysis laid out in the next Fig. 56.
The values of $c$ can be determinate using the approach shown in paragraph 4.4.3.1 afterwards:

$$V_c = K_v \sqrt{f_{\text{m}}' \cdot b \cdot d}$$

$$K_{v,1} = \frac{2 \cdot \rho \cdot E_f}{90 \beta_f' f_{\text{m}}'}$$

$$K_{v,2} = \frac{\rho \cdot E_f}{\rho_c \cdot A}$$

$$K_{v,3} = 2$$

$$K_{v,4} = \frac{c}{d}$$

Finally, the factored shear capacity is $\phi V_n = \phi V_{c,f} = 0.85(213.2) = 189.9$ kN/m ($0.85(14.61) = 13.01$ kip/ft) larger than $V_u = 160.5$ kN/m ($11.0$ kip/ft).
**4.4.2.3 Temperature and Shrinkage Reinforcement**

GFRP reinforcement perpendicular to the main flexural reinforcement is required to control both crack width and shrinkage of the concrete. The equation adopted by ACI 440 can be written as follows:

\[
\rho_{f,ss} = 0.0018 \frac{60,000}{f_{fu}} \frac{E_s}{E_f} \leq 0.0036 \quad (4.12)
\]

where \( f_{fu} \) (psi) is defined in Eq.(4.4), and \( E_s \) and \( E_f \) are the elastic moduli of steel and GFRP, respectively. The area of GFRP reinforcement deemed necessary for temperature and shrinkage can be expressed as follows:

\[
A_{f,ss} = \rho_{f,ss} bh \quad (4.13)
\]

and it is subdivided in two layers, each close to one of the concrete surfaces. In the previous equation, \( b \) and \( h \) represent unit width and height of the cross-section, respectively (\( b=30.5 \text{ cm, 12 in} \), and \( h=25.4 \text{ cm, 10 in} \)).

It is suggested to use a \( \phi 13 \) (#4) Aslan 100 GFRP bar spaced at \( \approx 30 \text{ cm (12 in)} \) center-to-center, as depicted in Fig. 57 representing a cross sectional view of the bridge deck.

Fig. 58 shows a longitudinal view of the prestressing CFRP tendons.
Fig. 57 - Bridge Internal FRP Reinforcement: Section at Mid-Span

Fig. 58 - CFRP Prestressing Tendons
4.4.3 Serviceability

Unlike steel reinforced concrete sections, members reinforced with FRP bars have relatively small stiffness after cracking. Therefore, serviceability requirements like crack width and long-term deflection need to be specifically tailored for composite structures as highlighted in ACI 440. In the following two sections, both crack width and long-term deflection checks will be presented.

4.4.3.1 Crack Width

The service moment per unit strip of slab deck can be calculated starting from the data of Table 4 as follows:

\[ M_s = M_{DL} + 1.3M_{LL} = 1.381 + (1.3)(6.549) = 44kN/m \text{ (m = 9.9 k - ft / ft)} \] (4.14)

where \( M_{DL} \) and \( M_{LL} \) represent moment due to dead and live load, respectively.

Crack width of FRP reinforced flexural members can be expressed as suggested by ACI 440 as follows:

\[ w = \frac{2200}{E_f} \beta k_b f_f \sqrt{d_c A} \] (4.15)

where \( \beta \) is the ratio of the distance from the neutral axis to extreme tension fibre to the distance from the neutral axis to the center of tensile reinforcement, \( k_b \) is a bond-dependant coefficient equal to 1.2, \( f_f \) represents the stress at service in the FRP, \( d_c \) is the thickness of the concrete cover measured from extreme tension fiber to the center of the bar, \( A \) is the effective tension area of concrete defined as the area...
of concrete having the same centroid as that of tensile reinforcement, divided by the number of bars \((N=2)\), and \(E_f\) is the modulus of elasticity of FRP. The above mentioned terms, can be written as follows:

\[
\beta = \frac{\beta_i h - c}{\beta_i d - c}
\]

\[
d_c = h - d
\]

\[
A = \frac{2d_i b}{N}
\]

Assuming a cracked concrete cross-section at service as shown in Fig. 59, the stress in the GFRP, \(f_f = E_f \epsilon_f\), can be calculated by solving the following system of equations:

\[
\begin{align*}
\frac{1}{2} E_c \epsilon_c bc - A_p E_p \epsilon_p - A_i E_i \epsilon_i &= N_{pi} \\
\frac{1}{2} E_c \epsilon_c bc \left(\frac{h}{2} - \frac{c}{3}\right) + A_p E_p \epsilon_p \left(d_p - \frac{h}{2}\right) + A_i E_i \epsilon_i \left(d - \frac{h}{2}\right) &= M - M_{pi}
\end{align*}
\]

where \(M_{pi}\) and \(N_{pi}\) represent the axial load and the bending moment to the centroid of the section, induced by the post-tensioning of the CFRP bars.

By solving Eq. (4.17) and finding the unknown \(\epsilon_f\), the following value for the stress in the GFRP can be evaluated as follows, after:

\[
f_f = E_f \epsilon_f = 13.58\text{MPa} (=1.97\text{ksi})
\]
The crack width calculated with Eq. (4.15) yields $w=0.086 \text{mm} (0.0034 \text{ in})$ which is smaller than the allowed values suggested by ACI 440 and equal to $0.51 \text{mm} (0.02 \text{ in})$.

4.4.3.2 Long-Term Deflections

Based on the conservative deflection analysis carried out on Section B.3.3, the long-term deflection can be calculated as suggested by ACI 440 as follows:

$$
\Delta = \Delta_{LL} + \lambda (\Delta_{DL} + 0.2 \Delta_{LL}) \quad (4.19)
$$

where $\lambda=1.2$ represents the multiplier for additional long-term deflection as recommended in ACI 440. Assuming the concrete cross-section uncracked because of the presence of the prestressing tendons, both concrete modulus of elasticity and moment of inertia can be expressed as follows:

$$
E = 57000 \sqrt{f_c} \\
I = \frac{bh^3}{12} \quad (4.20)
$$

where $b \sim 30 \text{ cm} (=12 \text{ in})$ for dead load analysis, and $b=E=1.4 \text{ m} (4.6 \text{ ft})$ for live load analysis. Eq. (4.19) yields to $\Delta=1.30 \text{ mm} (0.051 \text{ in})$
smaller than the suggested AASHTO value of $\frac{1}{800}=3.81\text{mm} \ (0.15\ \text{in})$.

4.4.3.3 Slab Creep Rupture and Fatigue

To avoid creep rupture of the FRP reinforcement under sustained loads, the stress level in the FRP bar should be limited to the value suggested in ACI 440. Specifically, when GFRP reinforcement is used, the stress limit has been set to be equal to $0.20\ f_{yu} \sim 87\ MPa \ (12.6\ ksi)$. The stress at service in the FRP can be found as:

$$f_s = \frac{M_s}{A_f\left(d - \frac{c}{3}\right)} = \frac{(9.9)(12)}{0.916\left(8.125 - \frac{0.943}{3}\right)} = 114.5\ MPa (= 16.6\ ksi) \quad (4.21)$$

Once again, because the bridge is mostly uncracked at service due to the prestress, the above findings are conservatives, and the value obtained from Eq. (4.21) can be considered acceptable.

4.5 Load Rating

Bridge load rating calculations provide a basis for determining the safe load carrying capacity of a bridge. According to the Missouri Department of Transportation (MoDOT), anytime a bridge is built, rehabilitated, or reevaluated for any reason, inventory and operating ratings are required using the Load Factor rating. All bridges should be rated at two load levels, the maximum load level called the Operating Rating and a lower load level called the Inventory Rating. The Operating Rating is the maximum permissible load that should be allowed on the bridge. Exceeding this level could damage the bridge.
The Inventory Rating is the load level the bridge can carry on a daily basis without damaging the bridge.

The Operating Rating is based on the appropriate ultimate capacity using current AASHTO specifications (AASHTO, 1996). The Inventory Rating is taken as 60% of the Operating Rating.

The vehicle used for the live load calculations in the Load Factor Method is the HS20 truck. If the stress levels produced by this vehicle configuration are exceeded, load posting may be required.

The tables below show the Rating Factor and Load Rating for this bridge. The method for determining the rating factor is that outlined by AASHTO in the Manual for Condition Evaluation of Bridges (AASHTO, 1994). Equation (4.22) was used:

\[
RF = \frac{C - A_1 D}{A_2 L (1 + I)} \tag{4.22}
\]

where: RF is the Rating Factor, C is the capacity of the member, D is the dead load effect on the member, L is the live load effect on the member, I is the impact factor to be used with the live load effect, \(A_1\) is the factor for dead loads, and \(A_2\) is the factor for live loads. Since the load factor method is being used, \(A_1\) is taken as 1.3 and \(A_2\) varies depending on the desired rating level. For Inventory rating, \(A_2 = 2.17\), and for Operating Rating, \(A_2 = 1.3\).

To determine the rating (RT) of the bridge Equation (4.23) was used:

\[
RT = (RF) W \tag{4.23}
\]

In the above equation, W is the weight of the nominal truck used to determine the live load effect.
For the Southview Bridge, the Load Rating was calculated for a number of different trucks, HS20, H20, 3S2, and MO5. The different ratings are used for different purposes by the bridge owner. For each of the different loading conditions, the maximum shear and maximum moment were calculated. Impact factors are also taken into account for Load Ratings. This value is 30% for the Southview Bridge. The shear and moment values for the deck are shown in below in Table 8.

<table>
<thead>
<tr>
<th>Truck</th>
<th>Maximum Shear (kN)</th>
<th>Maximum Moment (kN-m)</th>
<th>Maximum Shear with Impact (kN)</th>
<th>Maximum Moment with Impact (kN-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS20</td>
<td>15.1 (3.40 kips)</td>
<td>9.06 (6.55 k-ft)</td>
<td>19.66 (4.42 kips)</td>
<td>11.78 (8.52 k-ft)</td>
</tr>
<tr>
<td>MO5</td>
<td>14.7 (3.30 kips)</td>
<td>6.47 (4.68 k-ft)</td>
<td>19.66 (4.29 kips)</td>
<td>8.41 (6.08 k-ft)</td>
</tr>
<tr>
<td>H20</td>
<td>12.7 (2.86 kips)</td>
<td>5.93 (4.29 k-ft)</td>
<td>14.55 (3.72 kips)</td>
<td>7.70 (5.57 k-ft)</td>
</tr>
<tr>
<td>3S2</td>
<td>13.0 (2.93 kips)</td>
<td>5.89 (4.26 k-ft)</td>
<td>16.95 (3.81 kips)</td>
<td>7.66 (5.54 k-ft)</td>
</tr>
</tbody>
</table>

Table 9 below gives the results of the Load Rating pertaining to moment and Table 10 shows the results for shear. All calculations for the load rating are located in APPENDIX B.

<table>
<thead>
<tr>
<th>Truck</th>
<th>Rating Factor (RF)</th>
<th>Rating (RT) (Tons)</th>
<th>Rating Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS20</td>
<td>1.793</td>
<td>64.5</td>
<td>Operating</td>
</tr>
<tr>
<td>HS20</td>
<td>1.074</td>
<td>38.7</td>
<td>Inventory</td>
</tr>
<tr>
<td>MO5</td>
<td>2.482</td>
<td>89.4</td>
<td>Operating</td>
</tr>
<tr>
<td>H20</td>
<td>2.330</td>
<td>46.6</td>
<td>Posting</td>
</tr>
<tr>
<td>3S2</td>
<td>2.345</td>
<td>85.9</td>
<td>Posting</td>
</tr>
</tbody>
</table>
Table 10 - Rating Factor for the New Slab (Shear)

<table>
<thead>
<tr>
<th>Truck</th>
<th>Rating Factor (RF)</th>
<th>Rating (RT) (Tons)</th>
<th>Rating Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS20</td>
<td>2.152</td>
<td>77.5</td>
<td>Operating</td>
</tr>
<tr>
<td>HS20</td>
<td>1.289</td>
<td>46.4</td>
<td>Inventory</td>
</tr>
<tr>
<td>MO5</td>
<td>2.217</td>
<td>81.2</td>
<td>Operating</td>
</tr>
<tr>
<td>H20</td>
<td>2.202</td>
<td>44.0</td>
<td>Posting</td>
</tr>
<tr>
<td>3S2</td>
<td>2.148</td>
<td>78.7</td>
<td>Posting</td>
</tr>
</tbody>
</table>

Since the factors RF are greater than 1 then the bridge does not need to be load posted. In addition, from Table 9 and Table 10 the maximum operating and inventory load can be found as 64.5T and 38.7T respectively.

4.6 Southview Bridge Drawings

The following drawings show the main characteristics of the Southview bridge deck, consistent with the previously tested specimens and with the design presented in this section.

Drawing 1 and Drawing 2 show a plan strip view with the transverse and longitudinal section of the deck, respectively. Drawing 3 details profile, section, top and bottom view of the slab; FRP reinforcement details at the midspan and at the supports are also shown.
5 SOUTHVIEW BRIDGE DECK INSTALLATION

5.1 Introduction

This section details the installation of Southview bridge deck, conducted by Master Contractors LLC, overseen and documented by the author as part of this thesis. As outlined in the first section, the slab is 25 cm (10 in) thick, 12 m (40 ft) long and 6 m (19.83 ft) wide. It is supported by three intermediate reinforced concrete vertical walls. The three span lengths are, from North to South, 2.89 m (9.49 ft), 3.38 m (11.10 ft) and 3.30 m (10.82 ft), respectively, on centers. The material and construction specifications are detailed in Appendix C.

5.2 Pre-construction Meetings

In order to ensure a successful completion of the project, extensive planning and collaboration was required between the different entities involved in the construction: the City of Rolla in charge of the construction of walls and abutments expansion and of the post-tensioning system which was optimized for the particular field application as part of a cooperation between University of Missouri Rolla (UMR) and the Rolla Technical Institute (RTI) in the person of Mr. Max Vath. A first meeting was held on Tuesday, July 27th, 2004 at the City of Rolla. This meeting was attended by representatives of the City: the
engineer, David Brown and the foreman construction, Mr. Bill Cochran; the contractor in charge of the construction of the slab, Mr. Jason Cox from Master Contractors LLC, and Dr. Antonio Nanni and the author from UMR.

The three main topics were discussed at this meeting concerning issues of constructability, materials, and structural concerns.

- **Constructability issues** dealt with the schedule of construction and access to the bridge deck during construction. The existing lane of the bridge was the only access that could be used to reach the construction site; therefore carefulness had to be paid to the trucks that continuously passed on the bridge in order to create a safe environment for the workers.

- **Material issues** dealt with quality assurance testing: the University of Missouri-Rolla volunteered testing concrete samples extracted during the pouring and the steel rebar.

- **Structural issues** dealt on the discussion of details such as laying GFRP bars on the central wall in order to create a connection between slab and walls, the walls had to have different height due to the presence of Neoprene pads only on two of the three walls, and the construction of the formwork supporting the slab by the City of Rolla.

The second pre-construction meeting was held at the construction site on Thursday, August 12th 2004. The meeting was attended by the same people present at the first one, with the addition of one the bridge deck designers, Dr. Nestore Galati from UMR.

The main topics pertained to concrete, formwork and barrier:
Concrete issues dealt with quality and required characteristics of concrete to be used for the construction of the deck. A high slump value was determined to be necessary and it had to be achieved with the addition of super-plasticizers in order to make sure the matching of an adequate strength in a short period time. The City of Rolla was in charge of providing the concrete.

Formwork: In order to facilitate the post-tensioning a formwork longer than the slab itself had to be built on both sides of the bridge (see Fig. 87). This exceeding part of the formwork would have been cut after the post-tensioning operations, thus not affecting the construction of two more box cells, scheduled after the installation of the deck.

Barrier: During the construction of the walls it was also decided to build the barrier on the “FRP side” of the bridge using GFRP rebars instead of steel rebars. Such decision would in-fact allow the comparison between the durability of the FRP and the steel barrier to be built on the opposite side of the bridge, as already discussed and shown before.

5.3 Substructure Construction

5.3.1 Footing and Floor Construction

As highlighted before, the erection of the substructure and the extension of the existing abutments and walls were performed by the City of Rolla employees prior to the slab construction.
The 21st of July the works started with the removal of the guardrail, the wetting of the basement and the digging of the weed on the “FRP side”, as shown in Fig. 60.

![Fig. 60 - Wetting of the Basement and Digging of the Weed](image)

The following days, after the excavation of the bridge site and the moving of soil, the suction of the water overflowing from the creek was the main operation to deal with; indeed the area where the footing had to be built (see Fig. 61) was often and very easily filled with water either due to rain or to a mizzle. In order to remove the stagnant water different devices were utilized, such as electric pumps or a pipe allowing the water to flow from the creek getting over the basin to the opposite side of the stream (see Fig. 62).
On July 27th, the formwork and the steel reinforcement for the footing were placed (see Fig. 63), then the following day, after removing the remaining water, the footing was poured (see Fig. 64). The concrete slump was found to be 10 cm (4in).
Fig. 63 - Footing Formwork

Fig. 64 - Pouring of the Footing

Fig. 65 shows the Slump Test and the casting of concrete cylinders used to determine the concrete strength. The footing was built by the end of the working day (see Fig. 66).
On July 29th the floor reinforcement was placed, after filling the voids between the footing beams with gravel (see Fig. 67).
As highlighted before several delays were caused by the bad weather conditions. Fig. 68 shows the site conditions after a thunderstorm. The casting of the floor was completed on the 3\textsuperscript{rd} of August.

![Fig. 68 - Water Overflowing and Casting of the Floor](image)

### 5.3.2 Abutments Construction

The abutments construction started after replacing the former abutments caps, since the existing ones were in very bad conditions. Then, the formwork and the reinforcement of the abutments were placed quickly (see Fig. 69). Fig. 70 shows the device used to tie the steel reinforcement together. August 5\textsuperscript{th} the new abutments were cast (see Fig. 71).
Fig. 69 - Laying of Abutments Reinforcement and Formwork

Fig. 70 - Use of the Steel Ties Gun

Fig. 71 - Casting of the New Abutments
5.3.3 Walls Construction

Prior to the walls construction, during the curing of the new abutments, the dowels for the walls were fixed on the existing floor; thus the first wall reinforcement and formwork were placed (as shown in Fig. 72).

The following Monday, August 9\textsuperscript{th}, the first wall was cast (see Fig. 73).

![Fig. 72 - Reinforcement and Formwork of the First Wall](image1)

The second wall to be built was the one close to the other abutment. In this way it was possible to reuse the first wall formwork. The central
wall formwork had to be lower than the others to allow the insertion of the GFRP bars anchoring the slab to that wall.

The construction of the central wall was complicated by the lack of room and the short distance between vertical steel rebars to which the GFRP rebars had to be tied (see Fig. 74 and Fig. 75). In addition a central groove on the top of the wall was made to strengthen the connection between wall and slab (see Fig. 76).

The prefatory part of the project was completed on August the 12th (see Fig. 77 and Fig. 78).
Fig. 76 - Making of the Notch

Fig. 77 - Central Wall

Fig. 78 - Completion of the Prefatory Part of Southview Project
5.4 Slab Construction

Prior to the construction of the slab all the material needed was computed. The bill of materials and equipment used for the installation of the slab is reported in the following section.

5.4.1 Bill of Material

Table 11 summarizes the amount of FRP reinforcement needed.

<table>
<thead>
<tr>
<th>No.</th>
<th>Size</th>
<th>Length</th>
<th>Mark</th>
<th>Location</th>
<th>Fiber Type</th>
<th>Bending Sketches</th>
</tr>
</thead>
<tbody>
<tr>
<td>26</td>
<td>f 9</td>
<td>16.15 m</td>
<td>C1</td>
<td>Deck</td>
<td>Carbon</td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>f 19</td>
<td>13.11 m</td>
<td>G1</td>
<td>Deck</td>
<td>Glass</td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>f 19</td>
<td>13.11 m</td>
<td>G2</td>
<td>Deck</td>
<td>Glass</td>
<td></td>
</tr>
<tr>
<td>43</td>
<td>f 13</td>
<td>5.79 m</td>
<td>G3</td>
<td>Deck</td>
<td>Glass</td>
<td></td>
</tr>
<tr>
<td>43</td>
<td>f 13</td>
<td>5.79 m</td>
<td>G4</td>
<td>Deck</td>
<td>Glass</td>
<td></td>
</tr>
<tr>
<td>140</td>
<td>f 9</td>
<td>0.97 m</td>
<td>G5</td>
<td>Deck</td>
<td>Glass</td>
<td></td>
</tr>
<tr>
<td>43</td>
<td>f 19</td>
<td>1.95 m</td>
<td>G6</td>
<td>Barrier</td>
<td>Glass</td>
<td></td>
</tr>
<tr>
<td>43</td>
<td>f 13</td>
<td>1.85 m</td>
<td>G7</td>
<td>Barrier</td>
<td>Glass</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>f 13</td>
<td>13.11 m</td>
<td>G8</td>
<td>Barrier</td>
<td>Glass</td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>f 19</td>
<td>1.70 m</td>
<td>G9</td>
<td>Deck</td>
<td>Glass</td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>f 9</td>
<td>1.24 m</td>
<td>G10</td>
<td>Deck-Wall</td>
<td>Glass</td>
<td></td>
</tr>
</tbody>
</table>

Moreover the following materials were used:

1. **Neoprene pads**: NEWLON 60 durometer molded neoprene for load-bearing applications. Considering ~15x10 cm² (6x4
in\(^2\) pads, 1.3 cm (1/2 in) thick, ~15 cm (6 in) from each other, each wall needed 24 pads. Since only two of the three walls needed the pads and accounting also for the two abutments, a total of 96 pads were needed (~1.5 m\(^2\), about 2300 in\(^2\)).

2. **Styrofoam**: ~6.6 m\(^2\) (71 ft\(^2\)) of styrofoam were required to place around the pads.

3. **Chairs**: ~1.5 m (5 ft) long bolsters at ~60 cm (2 ft) spaced were deemed necessary to support the bottom longitudinal GFRP bars, considering ~6 m (20 ft) of width and ~12 m (40 ft) of length of the slab, 4 sets of 20 chairs, namely 80 chairs were ordered.

4. **Ties**: 5000 plastic zip ties were used to tie the bars and the duct, in order to have a complete “steel free” structure.

5. **Plastic duct**: The bridge is ~13 m (43 ft) long, so a ~13.7 m (45 ft) long duct was necessary for each tendon, plus three ~1.5 m (5 feet) long pieces of duct coming out from the deck and inserted through a T connector to the previous duct in the way to inject the grout. Hence the needed amount was about ~18 m (60 ft) of ~2.3 cm (9/10 in) diameter plastic duct for each tendon for a total amount of ~400 m (1300 ft). Adding thirteen ~13.7 m (45 ft) long safety duct (plus two ~1.5 m long coming out pieces) plus a supply, the overall duct length was ~700m (2300 ft).

6. **T connectors**: 3 T connectors of ~2.5 cm (1 in) internal diameter for each tendon trace were provided, so as to create a straight duct with a 3\(^{rd}\) piece coming out in the way to inject the grout. 26 tendons request 78 T connectors, plus 26 T
connectors for the safety duct is 104 T connectors, with supplies, hence 110 T connectors overall.

7. **Injection grout**: The Sikadur 300 PT is the grout for injection that was used for the bridge. Every bag is about ½ cubic foot, thus considering 39 per ~13.7 m (45 ft) long duct ~2.3 cm (9/10 in) diameter with a f 9 (#3) tendon inside, ~0.26 m³ (9.35 ft³) of grout was ordered, and namely 20 bags of Sikadur 300 PT were provided.

8. **Concrete**: The ~12*6*0.25 m³ (40*20*0.83 ft³) slab required ~21 m³ (750 ft³) of concrete, with supplies.

In addition the following tools were used to pull the CFRP tendons:

1. **Stereophone packs**: two ~37*25*25 cm³ (12*10*10 in³) stereophone packs were assembled in order to house the tendons, at the same time providing enough room to push the wedges inside the chucks before cutting the post-tensioned bars. More details will be itemized afterwards.

2. **Chucks**: Pulling 13 tendons per time, 2 chucks for each tendon and two more after the two hydraulic jacks when the load was applied were deemed needful. Therefore 28 chucks were requested overall. Each chuck includes an outer steel cylinder, a four-piece steel wedge and an inner copper sleeve (The same type used for the specimens load test, see section 3).

3. **Steel Plates**: Each tendon needed two steel plates between the edges of the slab and the pulling machine. Thus twentiesix ~(10*10*1.3) cm³ (4*4*1/2 in³) steel plates were required. All the steel plates had a 1.9 cm (0.75 in) inner hole, in order to avoid excessive transversal movement of the tendons.
Additionally the following equipment was used for the prestressing operations:

- 2 hydraulic jacks (40 tons), to pull the tendons from each side of the slab.
- 2 load cells (~222 kN, i.e. 50 kips each), to monitor the applied load.
- 1 injection pump, to inject the grout inside the duct.
- 4 wedges hammers to push the wedges inside the chucks.
- Orange box, an easily transportable system that records the load, the strains and the deflections.
- Wooden Boards, to build a surface suitable for the post-tensioning, given the skewness of the slab.

5.4.2 Outline of tasks

The installation of the deck proceeded based on the following tasks:

- Partial embedding of anchoring GFRP bars into the central wall, already described before.
- Laying of neoprene pads on the two external walls and on the abutments.
- Setting of the formwork.
- Installation of bottom chairs, bottom longitudinal and transversal GFRP bars.
- Installation of top chairs, top longitudinal and transversal GFRP bars.
- Placing of the GFRP reinforcement for the barrier.
- Placing of ductwork, T connectors and CFRP bars inside the ductwork; setting of the safety ductwork.
• Pouring and curing of concrete.
• Post-tensioning of CFRP tendons.
• Injection of grout inside the ductwork.
• Cutting of the tendons outside of the slab after the curing of the grout.
• Cutting of the edges of the slab.

5.4.3 Investigation of the Slab Construction

On August 13\textsuperscript{th} the installation work started, but, given the lack of some hangers (see details in Fig. 79 and Fig. 80), only on August 17\textsuperscript{th} the activities could restart.

![Fig. 79 - Formwork Hangers Details](image)
Fig. 80 - Laying of the Formwork

After the installation of the formwork, on August the 18th the following operation was the gluing of the Styrofoam on the two abutments and on the two walls that didn’t have the GFRP anchoring rebars (See Fig. 81).

Fig. 81 - Gluing and Placing of Styrofoam

Hence the Neoprene pads were placed in the room created in the Styrofoam as shown in Fig. 82. The neoprene pads were used to avoid
restraining horizontally the slab and therefore to effectively post-tension it.

Fig. 82 also shows the plastic chairs that supported the bottom layer of GFRP mild reinforcement.

![Fig. 82 - Neoprene Pads and Plastic Chairs Details](image)

The placing of the FRP reinforcement was speed up thanks to the presence of many students from RTI, leaded by Mr. Harold Martin, who envisioned this as an opportunity to teach to his students new technologies (see Fig. 83 and Fig. 84).

![Fig. 83 - Laying of Bottom Layer of GFRP Rebars](image)
On August 21st the top layer was also laid after placing all the GFRP top chairs. Furthermore, according to the design reported in Fig. 85, also the GFRP rebars for the barrier were placed as shown in Fig. 86:

![Fig. 85 - Barrier Design](image-url)
A wooden board was built to make the slab surface perpendicular to the tendons. This operation was performed in order to ease the post-tensioning phase. An alternative solution would have been to build a skewed slab, but it would have implied the development of a more sophisticated tool to pull the tendons. The position of the wooden triangles was designed in order to have the two tendons full centering each board. After the post-tensioning, the slab portion coming out of the abutments had to be cut (see Fig. 87 and Fig. 88).
On August 23rd the plastic ducts were placed and tied to the GFRP rebars as prescribed in the design specifications. Thirteen additional
safety ducts were placed (as shown in Fig. 89 and Fig. 90), straight and carefully tied to avoid their floating on the liquid concrete during the pouring.

![Fig. 89 - Slab Section](image)

![Fig. 90 - Plastic Ducts Detail](image)

T connectors were used on each end (see Fig. 91) in order to allow the injection of the grout as specified in the construction specifications in Appendix D.
Strain gages were also attached on the GFRP bars (see Fig. 92) and they were positioned as shown in Fig. 93.
The bridge deck was poured by the City Workers on August 25\textsuperscript{th}. The pour began at 7.30 am and was finished at 9.30 am (see Fig. 94).

In order to let the concrete filling all the voids inside the FRP cage, a more liquid concrete was used (Slump Test = 9 cm, \~4.5 in.), with the addition of super-plasticizers which increase the slump without
detrimental effects on the concrete strength. A total of 16 concrete cylinders were also prepared (see Fig. 95). Fig. 96 shows the just poured slab.

Fig. 95 - Concrete Cylinders Execution and Slab Leveling

Fig. 96 - Poured FRP Bridge Deck
5.4.4 Post-tension of the Slab

After a week of curing of the slab, on September 1\textsuperscript{st} the CFRP tendons were post-tensioned. The pulling was achieved by means of the machine already used for the two test specimens. Modifications were required afterward in order to decrease its weight and improving its functionality. These improvements were accomplished with the help of Mr. Max Vath, a Rolla Technical Institute instructor. Moreover, some handles were joined to better and easily transport it from a tendon to another (see Fig. 97).

![Fig. 97 - Pulling Machine Before and After the Optimizations](image)

Fig. 98 shows the modified pulling device. It comprises an open steel box having enough room to push the wedges inside the chuck after pulling the tendon, an hydraulic jack to apply the pulling force, a round steel plate, a load cell to measure the load, a second plate and a second chuck. Fig. 99 details the terminal part of the pulling machine.
Before pulling the tendons, they were carefully cleaned from grout, grease or dust; then the wedges were pushed inside the external chuck in order to provide a grip, using a copper sleeve (shown in the 3rd section). The tendons were pulled by means of two hydraulic jacks, connected to two pumps using load steps of 13-22 kN (3-5 kip) per side. Also the use of a single pump with two jacks connected in series was considered, but it was found to be harmful for the tendons being
the applied load not uniform along the tendon. A special two-parts steel hammer was built to push the wedges into the barrel (see Fig. 100).

The applied load was measured using a data acquisition system (Orange Box) connected to a computer allowing to monitor the applied load in real time (see Fig. 101).

After reaching the desired load of 62.3 kN (14 kips), corresponding to the 45% of the maximum allowable load to avoid the breaking of the
tendons for the tight grip, the wedges were pushed inside the inner chuck, so the jacks were released, engaging in such way the inner chucks. At this point the pulling device could be removed by cutting the FRP bar with an electric saw (see Fig. 102).

Fig. 102 - Wedges Insertion and Cutting of the Tendon

In 2 weeks the first half of the tendons was pulled. Some delays occurred due to further changes of the pulling machine in order to better suit the slab surface: it was not always possible to have a surface perfectly perpendicular to the tendon.

Fig. 103 - First Set of Pulled Tendons
The injection of grout followed the pulling of the tendons and was carried out with the use of a pump, after sealing the chucks to avoid grout leaking (see Fig. 104).

Fig. 104  -  Grout Injection and Slots Locking

After 4 days of curing the inner chucks were removed drilling the tendons inside of those.
On October 7\textsuperscript{th} the second set of tendons was pulled after a stop intermission of two weeks for a scheduled load test that the contractor had to carry out in Wisconsin, then the works could restart; thus the grout was injected also in the safety duct.
Eventually the extra part of the slab added for the post-tension was cut using a big cutter, as shown in Fig. 105. On October 15\textsuperscript{th} the Southview bridge-deck was completed (see Fig. 106).
Fig. 105 - Cutting of the Slab Edge

Fig. 106 - Southview Bridge-Deck Completed
6 CONCLUSIONS

The present thesis deals with a new technology, explored with the literature review, validated experimentally and verified on the field; it is part of a series of collaboration activities between the University of Naples, “Federico II”, and the University of Missouri-Rolla, UMR.

The project developed is a further step in the study and use of FRP in civil engineering; in fact, the use of FRP both for mild reinforcement (Glass FRP) and for post-tensioned reinforcement (Carbon FRP) was required for the construction of a bridge deck in the City of Rolla, Missouri.

The objectives of the project at UMR were as follows:

1. Evaluate the feasibility, behavior, and effectiveness of the new deck system, showing how FRP, in the form of GFRP as passive and CFRP bars as active internal reinforcement, could be an excellent solution replacing the steel.

2. Provide analytical data in support of the enhanced shear capacity of the concrete slab due to the CFRP prestressing.

Informations on existing prestressing and non-prestressing FRP bridge decks have been given in the section two, so that they can be compared with the new deck system that is the subject of this thesis. A summary of the main research works on shear behavior of prestressed FRP was also given, showing the lack of research projects on the specific topic of this thesis.

Section three dealt with pre-construction investigations that were conducted on two specimens representing a deck strip 457 mm (18 in) wide and 7 m (23 ft) long, with the same geometry and amount of
reinforcement. They were built and tested one to investigate the flexural behaviour, the other one the shear behaviour. The two beams were constructed by the contractor peculiar for the project also allowing for his familiarization with the use of non-conventional materials. Their testing as continuous slabs over three supports allowed the validation of the design calculations in terms of flexure and shear capacities.

The specimens were reinforced using 3 f 19 (6/8 in) GFPR bars as top and bottom mat and 2 f 9 (3/8 in) CFRP bars as prestressed tendons. The position of the prestressed tendons was varied along the slab in order to match the moment demand. In addition, in order to reproduce the actual field conditions also f 13 (4/8 in) GFRP bars spaced 305 mm (12 in) on center were placed in the transversal direction as temperature and shrinkage reinforcement.

The experimental results were compared with the theoretical moment and shear capacities of the slabs computed according to ACI 440.1R-03 provisions and to the equation developed by Tureyen A. K. and Frosh R. J.

According to ACI 440, the shear capacity increased from 30.7 kN (6.9 kip) with mild reinforcement to 69.8 kN (15.7 kip) adding the post-tensioned reinforcement, while the flexural capacity increased from 76.0 kN-m (55 kip-ft) to 97.9 kN-m (72.2 kip-ft), showing that the prestressing material is mostly needed for shear purpose rather than for flexure.

On the basis of the experimental investigation, it can be concluded that GFRP bars as passive and CFRP bars as active internal reinforcement could represent a feasible solution replacing the steel reinforcement of concrete slab bridges.
The objective of section four has been to provide the structural analysis of the new FRP concrete bridge deck based on the AASHTO HS20-44 design truck and provide calculations for its design using a combination of non-traditional corrosion-resistant composites materials.

Two load configurations were considered, consistent with AASHTO Specifications, the HS20-44 design truck and the design lane; the concentrated load and uniform load were considered to be applied over a 3.0 m (10ft) width on a line normal to the center line of the lane. These loads were placed in such positions within the design lane as to produce the maximum stress in the member.

Since the design truck analysis produced the highest stresses on the slab, only moment and shear related to this analysis were considered.

The design of the internal FRP reinforcement was carried out according to the principles of ACI 440.1R-03, using the same percentage of reinforcement already used and tested in the experimental phase.

Consistent with section 3 it was noted that the prestressing force is most needed for shear purpose rather than flexure. In fact, if the prestressing action was not considered, the concrete contribution to the shear capacity of the slab would have been $V_c = 220.4 \text{ N/m} (=15.1 \text{ kips/ft})$, and the final factored shear capacity of the bridge would have been equal to $\phi V_n = 86.09 \text{ kN/m} \ll V_u = 148.9 \text{ kN/m} (\phi V_n = 5.9 \text{ kips/ft} \ll V_u = 10.2 \text{ kips/ft})$, that is the shear capacity of the slab with post-tensioning; therefore the post-tensioning allowed to increase the shear capacity of the slab more than 40%.
Section five detailed the installation of Southview bridge deck, focusing on the post-tensioning of the slab, the most considerable and crucial part of the project.

The slab is 25 cm (10 in) thick, 12 m long (40 ft) long and 6 m (19.83 ft) wide. It is supported by three intermediate reinforced concrete vertical walls. The three spans lengths are, from North to South, 2.89 m (9.49 ft), 3.38 m (11.10 ft) and 3.30 m (10.82 ft), respectively, on centers.

The erection of the substructure started the 21st of July; the extension of the existing abutments and walls was performed by the City of Rolla employees prior to the slab construction.

On August 17th the installation of the deck started, after the partial embedding of anchoring GFRP bars into the central wall, in order to provide a fix central section, avoiding the total slipping of the superstructure during the prestressing operations. Hence, after the laying of formwork, the neoprene pads on the two external walls and on the abutments were placed in order to allow the axial sliding of the middle fixed slab.

The installation of bottom and top longitudinal and transversal GFRP reinforcement was completed by the 21st of August, after which the GFRP reinforcement for the barrier was also placed on the “FRP side”.

Wooden boards were positioned in order to ease the post-tensioning operations, then the placing of the ductwork, the T connectors and the CFRP tendons inside the ductwork was carried out by August 23rd.

Before pouring, some strain gages were also attached on the GFRP bars of the slab, in order to allow checking the behavior of the slab in service.
The bridge deck was poured by the City Workers on August 25\textsuperscript{th}; after a week of curing, on September 1\textsuperscript{st} the CFRP tendons were post-tensioned.

The pulling and releasing of the tendons were the newest and the most critical parts of the project.

The pulling of the tendons was achieved by means of the machines already used for the two test specimens, with two hydraulic jacks and special chucks to provide the grip between the machine and the tendons themselves, as detailed in the third section.

On September 15\textsuperscript{th}, after some delays occurred due to further changes of the pulling machine in order to better suit the slab surface, the first half of the tendons had been pulled, hence the grout was injected inside their ductwork.

The same procedure was carried out for the second half of the tendons; hence all the tendons edges outside of the slab were cut after the curing of the grout.

On October 15\textsuperscript{th} the edges of the slab were also cut.

The main result of this project has been to show how FRP, in the form of GFRP as passive and CFRP bars as active internal reinforcement, could be a feasible solution replacing the steel reinforcement of concrete slab bridges, and specifically the enhanced shear capacity of the slab due to the CFRP prestressing.

Moreover, the following conclusions can be summarized:

- The real advantage in the use of FRP materials as reinforcement of a bridge deck is seen through its increased durability, mainly due to the absence of corrosion. To better show it, a barrier with GFRP reinforcement was built on the new “FRP side”, in order to have a comparison during time with the steel reinforced
concrete barrier on the opposite side, which was going to be built after the FRP bridge deck installation.

- 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.

- The installation of the bridge highlighted the fact that having an efficient system is as important as having the adequate components. As well, for a new technology its learning curve must be overcome before its applications can be conducted proficiently.

- There was an increase in productivity using the FRP materials in each stage of construction versus conventional steel. This is directly associated with a labor savings. While this labor savings does not offset the initial increase in material costs, it is hoped that will become a factor in lowering the total life cycle cost of the bridge.

- Further investigation must be done in this field in order to improve and refine the techniques related to the post-tensioning of bridge-decks using CRFP, its future development being certainly hopeful and helpful.