Field and Laboratory Performance of FRP Bridge Panels

D. Stone, A. Nanni, & J. Myers
University of Missouri – Rolla, Rolla, Missouri, USA

ABSTRACT: The objective of this research project is to examine the use of FRP materials for bridge construction. In particular, this project utilizes glass FRP (GFRP) honeycomb sandwich panels. The installation of three bridges, one comprised solely of the FRP panels and two comprised of the FRP panels supported by steel girders, is outlined. Furthermore, results of preliminary testing conducted on the FRP panels are summarized. An in-situ bridge load test was conducted; a comparison is made between the theoretical and experimental values of maximum deflection and a discussion of the load transfer between panels is presented. Additional testing was conducted on an identical bridge panel section in the laboratory; results in terms of a load-deflection diagram, ultimate load, and the failure mechanism of the beam are noted herein.

1 INTRODUCTION

The objective of this research project is to examine the use of FRP materials for bridge construction. In particular, this project utilizes glass FRP (GFRP) honeycomb sandwich panels as bridge panels and steel-supported bridge deck panels. It should be noted that this project is still in progress and that the results presented herein are preliminary.

In order to reach the research objectives for this project a series of investigations are envisioned. First, so as to outline the construction-related issues associated with the use of these materials, a set of three short-span bridges was installed, each bridge utilizing FRP materials in a slightly different way to demonstrate the versatility of the materials.

The second investigation also makes use of the constructed bridges; in-situ load tests of the bridges illustrate the behavior of the overall structures, both in terms of panel behavior and installation details (e.g., panel-to-panel connections). Moreover, as load tests are conducted over time, examination of their long-term performance under real environmental conditions becomes possible.

Finally, the third investigative series deals with the laboratory characterization of these materials. In each case, the overall panel behavior is investigated in addition to a characterization of the individual materials.

To date, the three bridges have been installed on the secondary road system in a residential neighborhood. Testing thus far has consisted of an in-situ bridge load test of the FRP panel bridge and a test of an identical section in the laboratory. This paper presents an overview of the installation of the three bridges as well as the results of the initial bridge load and laboratory testing conducted.

2 BRIDGE CONSTRUCTION PROJECT DETAILS

Three short-span bridges were constructed in St. James, Missouri, using FRP panels. St. James, a small community in central Missouri, is located approximately half way between St. Louis and Springfield. The bridges are located in a residential area; consequently traffic volumes over the bridge are low, estimated at approximately 600 vehicles per day with very limited truck traffic. The three bridges are located at St. Johns Street, Jay Street, and St. Francis Street. The first two bridges, St. Johns Street and Jay Street, were constructed using FRP bridge deck panels supported by steel girders. The third bridge, at St. Francis Street, is comprised solely of FRP panels.

Glass FRP materials are used to construct the FRP honeycomb sandwich panels. The phrase FRP honeycomb sandwich refers to the construction of the panels themselves, which are comprised of a core of corrugated FRP material “sandwiched” between two faces of solid FRP material. For the panels used in this project, the thickness of the individual corrugation layers is 2.29 mm (0.09 in), with these layers shaped to construct cells approximately 50.8 mm (2 in) by 101.6 mm (4 in). Figure 1 illustrates the composition of the panel core material. Kansas Structural Composites, Inc. manufactured and installed the bridges.
The bridges were designed to carry a standard HS20-44 (approximately 180-kN) truck loading with deflections within the requirements of the American Association of State Highway and Transportation Officials (AASHTO). The modulus of elasticity obtained from previous testing conducted by the manufacturer is an average modulus of elasticity for the entire cross-section and was recommended at $1.34 \times 10^4$ MPa ($1.94 \times 10^6$ psi). Furthermore, for the purposes of calculating the moment of inertia of the panels the hollow portions of core have been ignored and the member cross section approximated as an I-beam.

3 INSTALLATION OF THE ST. JOHNS STREET AND JAY STREET BRIDGES

Since the configuration of the St. Johns Street and Jay Street Bridges is so similar their installation will be covered in a combined section. It should be noted that in addition to the standard highway truck (HS20-44) loading the bridges were also designed for the dead load of the deck, which is approximately 0.72 kN/m² (15 lb/ft²) and 0.77 kN/m² (16 lb/ft²), respectively for the St. Johns Street and Jay Street Bridges.

The St. Johns Street Bridge, an FRP deck supported by steel stringers, is comprised of six lateral half-width panels, having a thickness of 130.2 mm (5.125 in) including a 9.5-mm (0.375-in) wearing surface of polymer concrete, and 7 built-up steel stringers of size W14 × 90. The overall span length and width of the bridge are 8.08 m (26.5 ft) and 7.77 m (25.5 ft), respectively.

The Jay Street Bridge is comprised of four longitudinal panels, having a thickness of 168.3 mm (6.625 in) including a 9.5-mm (0.375-in) wearing surface of polymer concrete, and 7 built-up steel stringers of size W14 × 90. The overall span length and width of the bridge are 8.23 m (27 ft) and 7.77 m (25.5 ft), respectively.

Installation of both bridges proceeded based on the following outline of tasks:

- Install steel girders – drill holes into the abutment for the anchor bolts; place bearing pads and plates; anchor the plates to the abutment with the anchor bolts; place girders and weld them to the plates; install steel diaphragms.
- Install FRP decks – place panels onto the steel girders; attach panels together and secure to the girders; secure girders/panels to the abutments; fill the panel joints.
- Install guardrails – connect guardrail posts to the girders; attach guardrail to posts.

Tasks unique to the use of steel-supported FRP panels that warrant further attention are the connection of the panels to one another and to the girders and the connection of the panels/girders to the abutments. The connection of the panels together and to the girders was achieved through the use of a GFRP tube inserted between adjacent panels. The panels are mechanically clamped to the girders at the intersection of the panels and girders in eight locations and nine locations for the St. Johns Street and Jay Street Bridge, respectively. A detail of the panel joint and girder clamp connection for the St. Johns Street Bridge is illustrated in Figure 2; a similar configuration was used for the Jay Street Bridge, with slight modification due to the alignment of the girders and panels.

Figure 2. Cross Section of Panel Joint and Clamp Assembly.

The clamping assembly used on the St. Johns Street Bridge consists of two bolts extended down through the tube, one on either side of the girder flange, two steel plates one on the top and one on the bottom of the FRP tube, and small steel angles, which clamp under the girder flange to secure the panels to the girders. The clamping assembly for the Jay Street Bridge consists of a steel plate that has two bolts affixed to the top of the plate and four
bolts affixed to the bottom of the plate. The plate rests on top of the girder, while the two bolts on the top of the plate extend up through the tube and the four bolts on the bottom of the plate extend down around the girder flange, two on either side. Small angles placed onto the four bolts are the means of securing the panels to the girders.

Once the panels are clamped down to the girders, the connection of the panels/girders to the abutments is made. This connection consists of a steel T-beam installed over the edge of the panel, which is welded to the girder and also to the corrugated sheet piling, which is installed roughly 0.45 m (1.5 ft) below the top of the abutment perpendicular to the direction of traffic along the abutment. Figure 3 illustrates this detail, which serves to restrain the ends of the panels against vertical movements.

Figure 3. Cross Section of Abutment Detail.

Following the installation of the clamps and the connection of the panels/girders to the abutments, the joint between the panels is sealed using FRP strips and the same polymer concrete used for the wearing surface. The installation of the St. Johns Street and Jay Street Bridges took place concurrently with the setting of the first panels taking place on September 25, 2000. Installation was completed on October 4, 2000 and both bridges were opened to traffic on October 6, 2000.

4 INSTALLATION OF THE ST. FRANCIS STREET BRIDGE

The St. Francis Street Bridge is a prefabricated FRP slab bridge, consisting of four FRP panels, each 600.1 mm (23.625 in) thick, including a 9.5-mm (0.375-in) wearing surface of polymer concrete. The overall span length of the bridge is 8.00 m (26.25 ft) with a bridge width of 8.33 m (27.33 ft). The bridge was designed according to AASHTO deflection requirements for a standard highway truck (HS20-44) loading and also for the dead load of the panels, which is approximately 1.72 kN/m² (36 lb/ft²).

Installation proceeded based on the following outline of tasks:
- Install FRP panels – place panels onto the abutments; attach panels together; secure the panels to the abutments; fill the panel joints.
- Install the bridge guardrails – drill holes through the panels; attach the guardrail posts to the panels; attach guardrail to posts.

Tasks unique to the use of FRP panels that warrant further attention are the connection of the panels to one another, the connection of the panels to the abutments, and the connection of the guardrail posts to the panels.

The connection of the panels together and to the abutments was achieved through the use of GFRP tubes connected together and inserted between adjacent panels. A bolt inserted through the tubes in the joint was secured at the top of the tubes by a steel plate and at the bottom of the panel to a steel angle bolted to the abutment. Additionally, the panels are supported by the abutment itself. A detail of the panel joint and abutment connection are illustrated in Figures 4-5, respectively.

Figure 4. Cross Section of Joint.
5 LOAD TEST OF THE ST. FRANCIS STREET BRIDGE

The first load test of the St. Francis Street Bridge was conducted on March 9, 2001. The deflection of the bridge was monitored using direct current variable transformer (DCVT) transducers installed underneath the bridge and strain gages were applied to the bottom surface of the panels to monitor strains at the surfaces of the panels. The location of the DCVT transducers is illustrated in Figure 7, with the symbol denoting each individual instrument. The readings from the strain gages will not be discussed herein.

Loading of the bridge was accomplished with a loaded tandem-axle dump truck placed at various locations on the bridge. Table 1 outlines the truck’s axle spacing. The total weight of the truck was 242.17 kN (54,440 lb) with 76.69 kN (17,240 lb), 81.58 kN (18,340 lb), and 83.90 kN (18,860 lb), on each of the three axles from the front to the rear of the truck, respectively.

<table>
<thead>
<tr>
<th>Center-to-center spacing (m)</th>
<th>Out-to-out spacing (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WIDTH</td>
<td></td>
</tr>
<tr>
<td>Front axle</td>
<td>2.02</td>
</tr>
<tr>
<td>Middle axle</td>
<td>1.87</td>
</tr>
<tr>
<td>Rear axle</td>
<td>1.87</td>
</tr>
<tr>
<td>LENGTH</td>
<td></td>
</tr>
<tr>
<td>Front axle to Middle axle</td>
<td>4.60</td>
</tr>
<tr>
<td>Middle axle to Rear axle</td>
<td>1.35</td>
</tr>
</tbody>
</table>

Several passes of the truck were made, each at a different transverse position on the bridge; the results...
discussed herein are from the pass with the wheels centered on Panels 2 and 3. During each pass the truck was stopped at six longitudinal locations. Table 2 details the location of the truck stops. Due to the axle loads and axle spacing of the loading truck, truck location 6 corresponds to the worst-case loading condition. See Figure 8 for a schematic of the truck in position 6.

Table 2. Longitudinal Truck Locations.

<table>
<thead>
<tr>
<th>Stop</th>
<th>Truck Position</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>front axle of the truck approximately 0.61 m (2 ft) onto the bridge from the east end</td>
</tr>
<tr>
<td>2</td>
<td>front axle of the truck approximately 2.13 m (7 ft) onto the bridge from the east end</td>
</tr>
<tr>
<td>3</td>
<td>front axle of the truck at mid-span, approximately 3.66 m (12 ft) onto the bridge from the east end</td>
</tr>
<tr>
<td>4</td>
<td>front axle of the truck approximately 5.18 m (17 ft) onto the bridge from the east end</td>
</tr>
<tr>
<td>5</td>
<td>front axle of the truck approximately 6.71 m (22 ft) onto the bridge from the east end</td>
</tr>
<tr>
<td>6</td>
<td>middle axle of the truck at mid-span, approximately 3.66 m (12 ft) onto the bridge from the east end</td>
</tr>
</tbody>
</table>

Figure 8. Schematic of Truck Position 6.

The results of the load test are presented in Figures 9-10. It should be noted that the solid vertical lines represent the location of the panel joints and the dashed vertical line represents the location of load application. Moreover, the progression of the deflected shape from the top curve to the bottom curve represents the movement of the truck from position 1 though position 6.

Based on the material properties recommended by the manufacturer the theoretical mid-span deflection obtained via conjugate beam analysis is 4.8 mm (0.19 in). The mid-span deflection measured during the load test was approximately 3.96 mm (0.16 in) and 3.81 mm (0.15 in) for the two passes made. Possible causes for the smaller measured values could be variations in panel geometry due to the manufacturing process, variations in the locations of load application, stiffness contributions of the wearing surface and guardrails, and/or restraint provided by the connection to the supports and the soil backfill.

Based solely on the observation of the deflected shaped, it is evident that the joint behavior exhibited by the FRP panels is somewhere between the no transfer and perfect transfer behavior. For perfect transfer the curvature of the deflected shape should be the same on either side of the joint; Figures 9-10 illustrate the change in curvature from one side of the joint to the other. Future analysis will attempt to quantify the load transfer between panels by considering the data from other load passes conducted, but not discussed herein.

6 LABORATORY TEST OF A ST. FRANCIS STREET BRIDGE SECTION

Testing of a 577.1 mm (22.72 in) by 590.0 mm (23.23 in) section representative of the St. Francis Street Bridge was conducted under four-point bending. The 4.27-m (14–ft) specimen was tested over a clear span of 3.96 m (13 ft) with the equal loads applied approximately 1.52 m (5 ft) from each support, leaving a constant-moment region 0.91 m (3 ft) in length.

Measurements of the cross section were taken to better approximate the moment of inertia and the material properties recommended by the manufacturer were utilized. Based on simple beam theory and a failure criterion of a maximum fiber stress of 67.76 MPa (9825 psi), the failure load was approxi
mated at 667.26 kN (150,000 lb). The maximum stress failure criterion of approximately 67.76 MPa (9825 psi) was based on previous testing conducted by the manufacturer; it was also used during the design phase of the project. Another failure criterion prescribed by the manufacturer indicated a span-to-deflection ratio of approximately 100, which would have placed the failure load as high as 1089.86 kN (245,000 lb).

Instrumentation of the beam consisted of eight linear variable differential transformer (LVDT) transducers placed one on either side of the section at each support point, at the mid-point, and at one quarter-point of the beam. Additional instrumentation consisted of strain gages bonded to the core material and to the top and bottom faces of the section at mid-span; the data from the strain gages are not presented herein.

The beam was tested to failure through the application of quasi-static load cycles. First the beam was loaded up to approximately 20% of its predicted ultimate capacity and unloaded to approximately 22.24 kN (5000 lb); this cycling was repeated up to approximately 35% and 50% of the ultimate capacity and then the beam was loaded up to failure. Figure 11 illustrates the load-deflection diagram for the failure load cycle only. It should be noted that the solid line represents the theoretical behavior and the line outlined with points represents the experimental behavior of the section. Good agreement between design and experimental material properties is exhibited.

Failure of the beam was observed at approximately 862.99 kN (194,000 lb). The corresponding mid-span deflection was 33.78 mm (1.33 in), which yields a span-to-deflection ratio of approximately 115. Additionally, the maximum bottom fiber stress at failure was approximately 86.21 MPa (12,500 psi), 30% higher than the design failure limit.

The failure mode anticipated, based on experience from previous testing conducted by the manufacturer, was delamination between the top face and the core material. However, the failure mode exhibited by the specimen was delamination of the bottom face from the core material. This failure initiated at one end of the beam and progressed toward mid-span as the load was increased. See Figure 12 for a picture of the failure.

![Figure 12. Failure of the Specimen.](image)

7 CONCLUSIONS REGARDING TESTING OF THE ST. FRANCIS STREET BRIDGE PANELS

The conclusions presented herein are based on testing conducted to this point and pertain only to the St. Francis Street Bridge panels. The following conclusions can be drawn:

- The average value for modulus of elasticity recommended by the manufacturer yields good agreement between theoretical and experimental results for both in-situ and laboratory testing.
- Further examination of the in-situ bridge load test results are necessary to quantify the load transfer between panels, however load transfer between panels is evident from the deflected shape of the bridge.
- Further study of the laboratory test results needs to be conducted in order to quantify shear deformations that may have been experienced by the specimen.
- The failure mode exhibited by the laboratory specimen was delamination between the bottom face and the core material. The initiation of this failure occurred at the end of the panel.

Future research is planned to investigate the properties of the materials individually, the long-term performance of the FRP materials both in-situ...
and in the laboratory, and the behavior of the steel-supported FRP deck bridges and bridge deck panels. Further examination of the strain gage readings taken during the laboratory and in-situ tests should provide insight into the strain distribution in the core material and the maximum strain experienced by the section.

ACKNOWLEDGEMENTS

The authors would like to acknowledge funding and support received from the University Transportation Center at the University of Missouri – Rolla, the Missouri Department of Transportation, the City of St. James, the National Science Foundation, and the Missouri Department of Economic Development.

REFERENCES
