Experimental Investigation of GFRP Hybrid Full-scale Bridge Deck Panels Subjected to AASHTO Design Truck Wheel Load

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***Abstract*—This paper aims to demonstrate the feasibility of using a novel glass fiber-reinforced polymer (GFRP) sandwich panel in bridge deck applications. This new system integrates a polyurethane foam core that is sandwiched between two GFRP facings. With the purpose of investigating the performance of this new system, experimental static testing was performed. The investigation focused on the ultimate flexural strength capacity, stiffness, and panel-to-panel connection of the proposed bridge deck. The results of the flexural testing showed that the tested bridge deck panels exceeded the AASHTO Design Code Strength requirements by nearly three times. In addition, the bonded butt-type sandwich panel-to-panel connection transferred the loading between the panels beyond that required by code.**

***Index Terms*—Sandwich Panel, Glass Fiber Reinforced Polymer, Hybrid Bridge Deck.**

I. Introduction

In the past two decades, light-weight bridge deck systems fabricated from glass fiber-reinforced polymer (GFRP) composites have gained attention in civil infrastructure communities as an alternative to traditional construction materials - steel and concrete. This attraction is due to the fact that GFRP composites have high strength, light weight, rapid constructability, and excellent corrosion resistance [1]. Historically, most of the FRP composite bridge decks built in the United States are made of honeycomb cores. The high cost of honeycomb cores limited its use mainly to the aerospace industry. As a result, a number of studies have been conducted to propose inexpensive cores for civil engineering applications; ranging from polyvinyl chloride and polyurethane foam cores, foam cores with through thickness GFRP, GFRP rectangular holes filled with polyurethane foam, and through thickness-fiber core [2]-[7].

Fiber reinforced-polymer (FRP) composite sandwich panels typically span across the direction of traffic, so that the panels' joint connection must be designed to ensure the integrity of the panel and load transfer efficiency between the jointed panels. Joint connections consist of either mechanical fasteners, adhesive bonding, or a combination of both [8]. Previous research studies have shown that bolted connections of panels are more vulnerable to fatigue loads than adhesively glued ones [9], [10], while adhesively bonded connections have displayed more efficiency in transferring load between panels and having higher fatigue resistance [11].

The sandwich bridge deck panels of this study were produced by Structural Composites, Inc., Florida, using the vacuum assisted resin transfer molding (VARTM) process. These panels were composed of top and bottom GFRP facings consisting of twelve layers of E-glass fabric and corrugated webs consisting of four layers of E-glass fabric. The trapezoidal polyurethane foam keeps the top sheet, bottom sheet, and corrugated webs in position during manufacturing. The combination of the trapezoidal foam cores with FRP facings resulted in a very efficient cross section, as demonstrated in Fig.1.

Six full-scale, prototype sandwich bridge deck panels; three for flexural testing (Fig. 1) and three for panel-panel connection testing (Fig. 3) were considered in this study. The flexural testing panel has a total height of 9.25” (235 mm), width of 2’–5½” (749.3 mm), with a span length of 9’–2” (2.79 m), while the panel-to-panel connection testing panel has a total height of 9.25” (235 mm), width of 3’- 04½” (1028.7 mm), with a span length of 5’– 0¼” (1.53 m).

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This present study is an outcome of previous studies conducted by the authors [12]-[15] that proposed different core systems. This study focused on the implementation of the proposed bridge deck sandwich panel on actual bridges and addressing issues such as sandwich panel-to-panel connections.

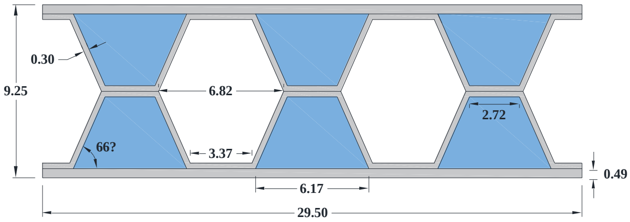

Figure 1. Overall sandwich panel bridge deck, all dimensions in inches (1 in. = 25.4 mm)

II. Experimental Program

Six full-scale, prototype bridge deck panels, manufactured using the VARTM process, were tested under flexural and panel-to-panel connection testing.

A. *Flexural Test Setup*

Three full-scale, prototype bridge deck panels were tested under a concentrated truck wheel load. The test setup is illustrated in Fig. 2. The truck wheel tire is simulated through a 20 in. (508 mm) by 10 in. (254 mm) steel plate, with the 20 in. (508 mm) dimension oriented perpendicularly to the traffic direction. The loading wheel load was placed at midspan of the bridge deck panel in order to maximize the flexural stresses throughout the testing. Vertical deflections and strains at the mid and the quarter-span points of the panels were measured using potentiometers and strain gauges, see Fig. 2. All of the instrumentation was connected to a data acquisition system.

Figure 2. Static flexural test setup

B. *Panel-to-panel Connection Test Setup*

Three full-scale, prototype bridge deck panel pairs (see Fig. 3 & 4) using a bonded butt-type joint were tested. The panels were adhered using the same methacrylate adhesive used to fabricate the full prototype panels. A photo of the test setup is shown in Fig. 4. The truck wheel tire is simulated through a 20 in. (508 mm) by 10 in. (254 mm) steel plate, with the 20 in. (508 mm) dimension oriented perpendicularly to the traffic direction. The loading point was placed at midspan of one of the bridge deck panels of the bonded pair. Vertical deflections were measured using potentiometers at the mid-span of the jointed connection. All of the instrumentation was connected to a data acquisition system.

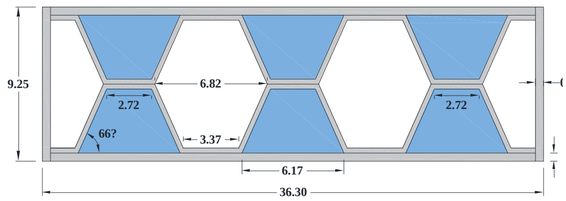

Figure 3. Static flexural test setup

Figure 4. Static flexural test setup for panel-to-panel joint connect

III. Test Results

A. *Flexural Tests*

The general behavior and response of the three bridge deck panels were very consistent. The panels behaved linearly during the course of loading (Fig. 5). A load-deflection curve is plotted for one of the specimens as displayed in Fig. 5. The load-deflection curve represents a linear behavior throughout the whole testing. The initial failure was local buckling of the top facing (Fig. 6b), and then followed by separation of the top facing from the corrugated webs along the specimen length (Fig. 6c). Investigating the specimens before reaching the final failure, which was separation of the top facing, did not show any visible damage. This concludes that no permanent deformation would be caused during overloading during the bridge service, i.e., these sandwich panels can support a load up to their failure point and return back to their original shape once the load is removed.

Looking at the load-deflection plot (Fig. 5) at the mid and quarter spans, it shows another essential characteristic, which is the transverse stiffness of the sandwich panel helped to involve the majority of the panel's cross section in supporting the applied point load.

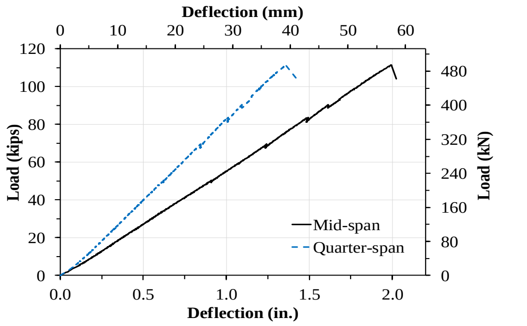

Figure 5. Load-deflection curve at mid-span and quarter-span of the panel

Another important aspect for sandwich panel bridge decks that should be mentioned is the deflection under serviceability loading. The design of FRP sandwich deck panels is usually limited by deflection due to the fact that the FRP materials have high strength to stiffness ratios. As can be seen from Fig. 5 that the average deflection of the three tested panels is approximately 0.41 in. (10.44 mm), which corresponds to a service truck load of 37.2 kips. This deflection exceeds the AASHTO L/800 deflection limit criteria [16]. However, the deflection value would be reduced in an actual bridge as the sandwich panel would be continuous over several girder beams, while the tested laboratory panels were simply supported deck panels. In addition, this deflection limit has been recently reduced to L/500 for FRP bridge deck panels [17].

(a) Deflection during flexural loading, 75 kip

(b) Initial wrinkling of compression facing

(c) Separation of compression facing from webs

Figure 6. Failure modes of flexural testing

B. Panel-to-panel Connection

The overall behavior of the three bridge deck panels was very consistent. As can be seen in Fig. 7, the load-deflection plot displayed linear behavior with nonlinearity behavior only in the last 30 percent of the loading. The initial failure was triggered by local buckling of the webs underneath the concentrated load point (Figure 8). The nonlinearity behavior at the higher loading suggests some inelasticity of the sandwich panel-to-panel joint, which could be a minor loss of load sharing between the two sandwich panels. It could be due to the inelastic behavior of the methacrylate adhesive which was used to glue the two panels, reducing the rigidity of the joint. In addition, the panels were thoroughly inspected during loading and no sign of damage, cracks, or distress were reported.

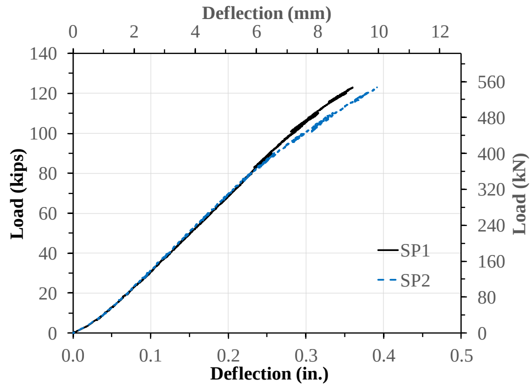

Figure 7. Load-deflection curves at each panel edge adjacent to the bonded joint

The bonded joint was able to transfer the loading beyond that required by code. It is also noted that the panel-to-panel joint did not fail in terms of strength during the full range of loading. Figure 8 demonstrates the bearing failure, as a result of web buckling underneath the point load.

Figure 8. Bearing Failure underneath the point load

IV. Summary and Conclusions

Three full scale, prototype, full-scale bridge deck panels were tested in flexure and another three bonded butt-type panel-to-panel connection panels were also tested. For the flexural tests, the results showed that the panels considerably exceeded the AASHTO Design Truck Factored Wheel Load by approximately three times, signifying the level of safety for this proposed bridge deck panel system. As for testing the bonded butt-type panel connection, the results showed the load was 100% transferred, up to twice the AASHTO standard load. The panels failed due to local buckling of the sloping webs underneath the wheel load.

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