Testing and Monitoring of a Five-Span Bridge with Fiber Reinforced Polymer Deck Systems

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ABSTRACT

As part of a project on the application and constructability of fiber reinforced polymer (FRP) bridge decks, four different deck systems were installed in a three-lane, five-span bridge in Dayton, Ohio. The spans range from 37 to 45 m, and the bridge consists of six steel girders. The span lengths and large number of panels created a unique opportunity to evaluate the response of four common FRP bridge deck systems under identical traffic and environmental conditions. The performance of the bridge has been monitored through field documentations, long-term continuous monitoring of key responses, controlled static and dynamic truck load tests conducted on a regular basis, and multi-reference modal tests performed in conjunction with the truck load tests. The measurements from nearly 300 sensors have allowed a detailed evaluation of critical design issues such as the behavior of FRP panel-steel girder connections and connections between FRP panels, impact factor and distribution factors for bridges with FRP decks, thermal characteristics of FRP panels versus conventional reinforced concrete decks, critical role of thermal behavior of FRP panels on the overall performance, level of composite action, and serviceability issues for bridges with FRP decks. This paper provides an overview of the characteristics of the different FRP deck systems, and highlights the important observations and test results.

INTRODUCTION

In recent years, fiber reinforced polymer (FRP) composite deck systems have become an alternative to traditional reinforced concrete (RC) slabs. The lightweight and non-corrosiveness of FRP panels are the main expected benefits of the application of the “new” materials in structures. Easy and quick installation of lightweight prefabricated panels, increased traffic capacity due to a better live load- dead load ratio, and less maintenance with a longer expected service life are the key advantages that bridge engineers are hoping to gain. In view of such advantages, FRP decks have been installed in a number of projects. However, these projects single-span bridges on small rural roads with spans rarely exceeding 10 m. The bridge used in the reported research offered a number of unique features that are expected to overcome the shortcomings of the previous projects. This bridge is located on a major route with an average daily traffic (ADT) of 10000 vehicles per day, which allows a detailed study of the influence of repetitive loadings. The size of the bridge permitted installation of multiple large panels from different manufacturers, provided an opportunity to examine design issues such as panel to panel and panel-stringer connections, and allowed a direct comparison of performance and examination of basic data for a number of FRP deck systems under identical loading and environmental conditions in one structure.

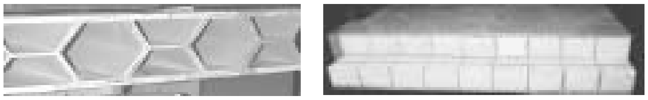
This paper presents the experimental data, as measured through controlled field tests and continuous response monitoring, and important observations to address specific issues related to construction of each panel type, connection details, short- and long-term responses, and key design parameters. Using field measurements and observations over a two-year period, general observations regarding application of FRP panels as bridge decks are made.

DESCRIPTION OF TEST BRIDGE

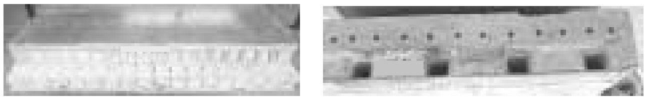
The focus of the reported project was around a pair of three-lane, five-span bridges located on Route 49 in Montgomery County in Dayton, Ohio (MOT-49-1.634). The twin bridges are called “Salem Avenue Bridge”, and span over the Great Miami River. The spans are 40, 42, 44, 42, and 40m, and the bridge is 14.6 m wide. The bridges were built in 1951. Each bridge consists of six built-up steel stringers. The original deck was a standard reinforced concrete (R.C.) deck, with a thickness of 20 cm. The deck had deteriorated to a level that it had to be replaced completely.

DESCRIPTION OF FRP DECK SYSTEMS

The selected decks include three “all FRP” decks and one hybrid FRP-concrete deck. Two panels (FRP2 and FRP3) are based on the concept of sandwich construction, the third one (FRP1) uses interlocking components, and the fourth system (FRP4) involves FRP panels used as stay-in-place form and tensile reinforcement. All the panels are 20 cm thick. The “all FRP” (FRP1, 2, and 3) panels cover the whole width of the bridge resulting in a panel length of 14.63 m. The width of each panel is 2.44 m, and the three “all FRP” systems have a unit weight of approximately 4.7 kN/m^3. The FRP4 panels, on the other hand, only cover one bay between the girders. Their length is 2.4 m and their width is 46 cm. The 20 cm thick concrete infill (with #5 GFRP top-reinforcing bars spaced 15 cm and 24 cm transversely and longitudinally) dominates the unit weight of these panels, and the deck



(a) FRP1 (b) FRP2



(c) FRP3 (d) FRP4

Fig. 1 Cross Section of Various Deck Systems is approximately five times heavier than the “all FRP” decks. A 9.5 mm thick polymer based overlay, acting as wearing surface, was applied on top of the all FRP panels. The concrete in-fill in the hybrid deck system (FRP4) provided the necessary riding surface.

INSTALLATION PROCEDURE

In order to facilitate field installation, compromises were made so that uniform panel-girder connection details would be used for the “all FRP” systems. Installation of FRP4 hybrid system was different from that for FRP1, 2, and 3 panels. FPR1, 2, and 3 panels were placed over latex modified concrete haunches, poured over the top flanges of the girders, to ensure proper elevation of the panels. The panels were bonded to the haunches by a layer of epoxy. After installation of all the panels, occasional gaps between the panels and haunches were pressure grouted to ensure full support underneath the panels. FRP1, FRP2, and FRP 3 panels were delivered with two predrilled stud holes per girder line. After installation of all the panels, studs were welded onto the girders through these holes. Note that pockets had been left in the haunches at the locations where studs would be welded. Before the application of polymer overlay, the stud-holes were filled with non-shrink grout. These studs were not intended to provide composite action.

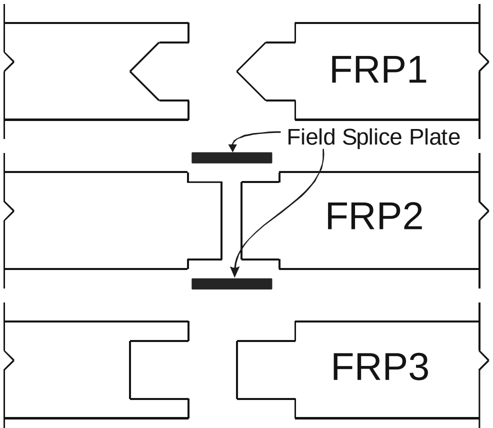
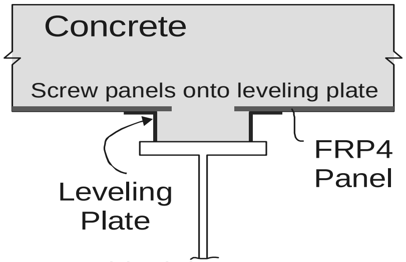
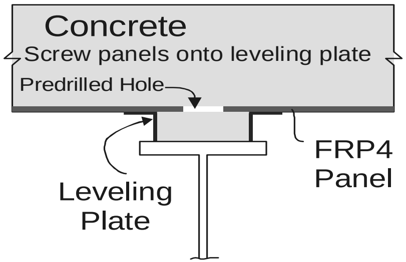


Fig. 2 Panel-Panel Connections

FRP1 and FRP3 panels were interconnected through a tongue and groove joint system, and FRP2 panels were bonded together by 0.76 cm thin field-splice plates, as shown schematically in Fig. 2. Installation of FRP4 panels was similar to conventional RC decks. These panels were secured by screws to leveling plates that had been welded to the top flange of the steel girders. As shown in Fig. 3a, the panels were not extended over the girders so that the concrete would fill the gap between the leveling plates. On the other hand, the panels cantilevered over the fascia girders and covered the area above the flanges as indicated in Fig. 3b. In order to allow concrete to flow and fill the gap between leveling plates, 10 cm holes had been predrilled in every other panel. Proper steps had been made to ensure that these holes would line up directly above the flanges. For the FRP 4 panels, field joints were not a major issue. Each panel, covering one bay, was screwed to the leveling plates, and was filled with 20 cm of concrete. The concrete in-fill interconnected the individual components of this system, and the resulting deck system was essentially similar to a monolithic reinforced concrete deck.



(a) Girders 2 to 5



(b) Girders 1 and 6

Fig. 3 FRP 4 Panel-Girder Connection

TEST PROGRAM AND RESULTS

Static and high-speed controlled truckload tests, multi-reference modal tests, and long-term monitoring of critical responses (recorded continuously at 30-minute intervals) were used to compare the performance of the new bridge with FRP panels, and to monitor possible changes of the performance of the FRP panels over a two-year period. Reference tests on the bridge with RC deck and shortly after installation of FRP panels established the baselines of the responses monitored during the next two years. For each deck system, one cross section of the bridge was instrumented. A detailed description of the sensors and testing program is provided elsewhere [3]. Using the responses measured during controlled tests and from continuous monitoring, a number of key data and observations are presented to characterize the performance of the bridge.

(a) Panel Movements

Transverse movement of the panels relative to the girders (i.e., perpendicular to the traffic direction) was very small. The movements of the panels relative to each other in the longitudinal direction, summarized in Table 1, are also small but more significant than the transverse panel-panel movements. Considering that the maximum longitudinal movement of FRP 2 panels during controlled truck tests with four loaded trucks (each weighing approximately 150 kN) was 0.25 mm, the movements in Table 1 are predominantly from environmental loading. The field joints are apparently adequate to resist expansions in the longitudinal direction.

Table 1. Maximum Average Panel - Panel Long. Movement

Panel	Movement (mm)
FRP1	0.4
FRP2	1.77
FRP3	1.23
FRP4	0.3

To overcome unexpected panel upward movements of FRP 2 panels during construction, the first two panels at each end of the section with these panels were held down through steel angles welded to the studs. At all other locations, the self-weight of the panels and sidewalk, and adhesive between the panels, if any, and haunches would oppose any upward movement of the rest of the panels. The additional dead load of the sidewalk reduced the vertical movement of girder 2, as seen from Table 2. The panel movement is generally larger for the girders farther away from the sidewalk. The locations of maximum movement are not consistent for various deck systems, e.g., FRP2 panels exhibit the largest movement at girder 4 while FRP3 panels show the largest vertical movement at girder 5. The maximum vertical movement of FRP 2 panels (4.3 mm) is at least 4 times larger than the movement experienced by any other panel. The differences in vertical panel movements and the locations of the largest movements are attributed to (a) the local variations in the quality of the support between the panels and haunches, (b) the differences in the thermal characteristics of various panels, and (c) the quality of the field joints. The influence of inadequate support was investigated by additional series of tests. In one series, a

Table 2. Max. Vertical Panel-Girder Movements in mm

Panel	Girder # 2	Girder # 3	Girder # 4	Girder # 5
FRP 1	0.38	0.46	0.38	0.38
FRP 2	0.76	3.3	4.3	2.29
FRP 3	0.5	0.47	0.89	1.02
FRP 4	0.37	0.38	0.35	0.25

Girder # 2 is closer to the side walk
Girder # 5 is closer to the median

single truck was oriented perpendicular to the traffic direction. The truck was initially positioned such that its back wheels were as close as possible to the sidewalk, and was moved toward the median in approximately 60 cm increments. The deflections of panels relative to the girders when the back axle of the truck was centered between girders 4 and 5 are shown in Fig. 4. The deflections relative to the interior girders were measured by a pair of displacement transducers mounted on each side of the girder.

The results in this figure indicate a downward movement of FRP2 and FPR3 panels, at least at the instrumented section, on both sides of the flanges of girders #4 and #5. Had the panels been supported properly, the panels would have lifted on one side of the flange and gone down on the other side of the flange. The trends of the measured deflection for these panels suggest improper support, at least at the instrumented section, at girders #4 and #5. During replacing of FRP2 and FRP3 decks, a few weeks after these tests, the supporting conditions of the FRP2 were visually inspected. A photograph of the support conditions for girders #5 and #6 after removing the deck is shown in Fig. 5. Over girder # 5, a gap between the panel and haunch was discovered. This gap is believed to be the main reason for the trends of the panel movements shown in Fig. 4. Other tests, in

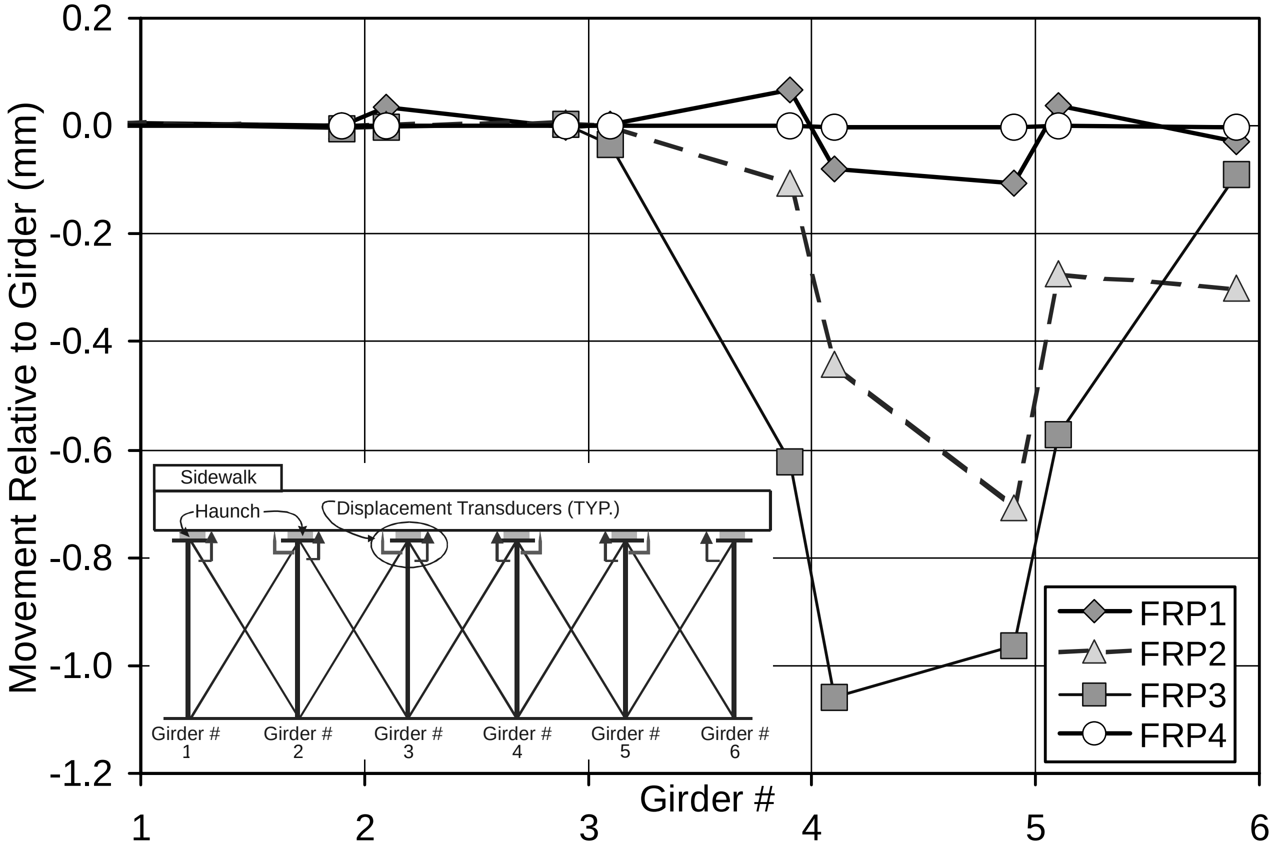


Fig. 4 Panel Movement Relative to Girder

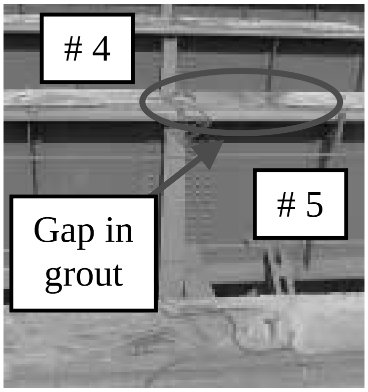


Fig. 5 Photograph of Support for Girder #5

which the longitudinal location of the truck was varied, also point to inadequate support for FRP2 and FRP3 panels [4]. The deflection profiles for FRP4 decks are in agreement with what is expected for proper support, i.e., the panel movements on either side of the girder are of opposite directions. FRP1 panels are apparently supported at all girders except for girder #2. On the right side of girder #2, the panel movement was unexpectedly large, suggesting improper support. In-situ pressure grouting after placement of FRP panels, in lieu of cast haunches, would most likely produce more even supports underneath the panels.

As explained above, another factor contributing to the different levels of panel movements are the differences in the thermal characteristics of the panels. Over two typical summer days, the measured difference between the top and bottom temperatures is shown in Fig. 6. FRP2 panels show the largest temperature gradient, as the foam cell cores apparently act as insulation between the bottom and the top skins of the panels. FRP1 panels experience vertical temperature differences as the double-trapezoid profile (see Fig. 1a) divides the panels into bottom and top cells; hence, convection within one single cell does not occur. The thermal conductivity and insulation characteristics of concrete in FRP4 prevented the development of extreme temperature differences. FRP 2 panels, which exhibit the largest thermal gradient, show the largest longitudinal and vertical movements. Therefore, the panel-girder connections need to be designed to account for thermal behavior of panels, and potential thermal stresses in the panels should be incorporated as part of the design, particularly for systems that can experience large thermal gradients. Moreover, possible influences of large thermal gradients on the integrity of face sheet-core connection should be considered in design and detailing of sandwich type panels.

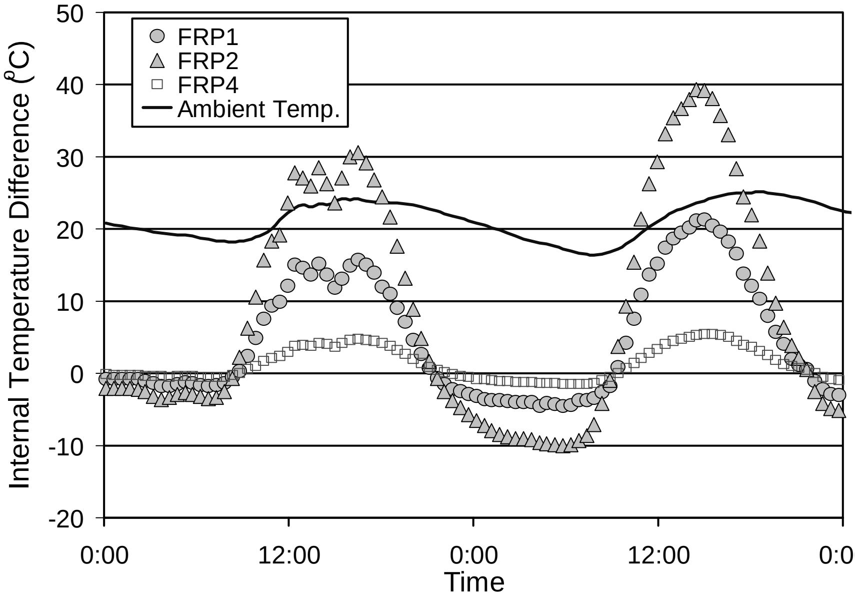


Fig. 6 Variation of Internal Panel Temperature

(b) Distribution Factors

The distribution factors were calculated by using the measured bottom flange strains [3]. The distribution factors along with the value recommended by American Association of State Highway and Transportation Officials (AASHTO) are shown in Fig. 7. The maximum distribution factors for each section are approximately equal or less than the AASHTO recommendation. The variations among different tests are attributed to slight changes in the truck positions. The values recommend by the AASHTO specifications seem to be also applicable for calculating the distribution factors for bridges with FRP decks.

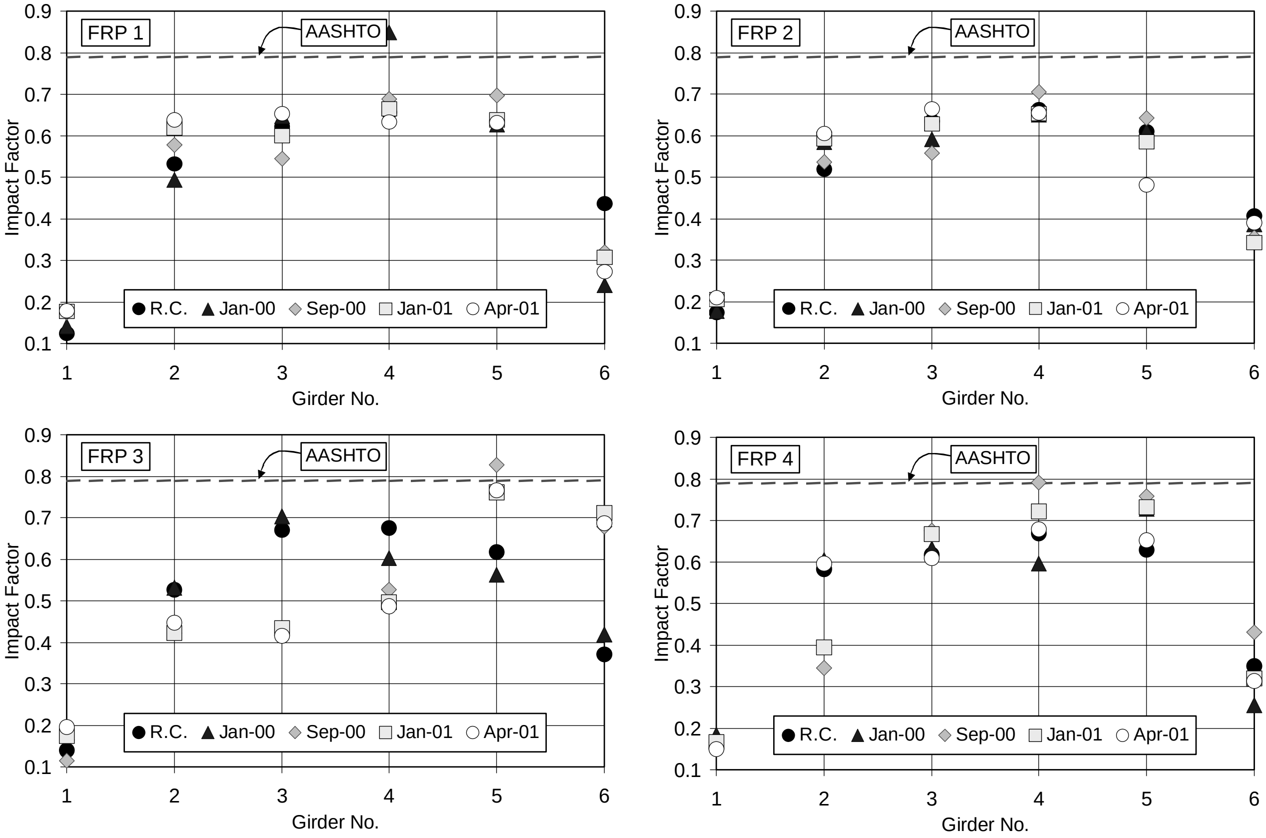


Fig. 7 Distribution Factors for FRP1, FRP 2, FRP3, and FRP4

(c) Impact Factors

The limit for impact factor in the standard specifications of AASHTO [1] is 0.3, and is 0.33 in the LRFD specifications [2]. Based on the AASHTO specifications, the impact factor for the test bridge is 0.2. These values of impact factors are not necessary valid for bridges with FRP decks. The average impact factors, computed based on a procedure discussed elsewhere [4] for the replacement decks from various stages of testing were found to be 0.17 for FRP4 (min. = 0.12, max. = 0.2), 0.23 for FRP1 (min. = 0.21, max. = 0.27), FRP3 (min. = 0.18, max. = 0.27) panels, and 0.37 for FRP2 system (min. = 0.28, max. = 0.42). With the exception of FRP2 panels, the experimental impact factors are within the range of the limits recommended by AASHTO.

SUMMARY AND CONCLUSIONS

A 207-m, five-span bridge was retrofitted with four different FRP deck systems. Field observations and long-term monitoring over a period of two years in addition to regular controlled tests allowed a detailed comparison between the performance of different panel systems under identical traffic and environmental conditions.

1) The values of girder distribution factor for all deck systems are generally comparable to the ones for the original RC deck.

2) Except for FRP2 decks, the impact factors for FRP deck systems are lower than the AASHTO limit. The smallest impact factor, however, is for the hybrid deck system FRP4.

3) The thermal characteristics of FRP panels are influenced by the detailing of the panel cross section in addition to the thermal properties of the individual components of the panels. Internal insulation of the core material or horizontal dividing layers may result in large thermal gradients that influence the overall panel behavior, such as panel movements. Design of the field joints and connections to the supporting members needs to account for the thermal behavior of FRP panels. Thermal behavior of FRP4 decks is similar to regular RC, and is not a major design issue.

4) The performance of FRP4 hybrid system was very satisfactory and least surprising. However, these systems lack some of the benefits offered by “all FRP” deck systems, such as dead load reduction and reduced construction time.

5) Proper support underneath “all FRP” deck systems is important to prevent panel movements.

ACKNOWLEDGMENTS

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REFERENCES

1) American Association of State Highway and Transportation Officials (AASHTO). (1996), “Standard Specifications for Highway Bridges,” Washington, DC.

2) American Association of State Highway and Transportation Officials (AASHTO). (1998), “LRFD Bridge Design Specifications – 2nd Edition,” Washington, DC.

3) Reising, R.M.W. et al. (2001), “Performance of a Five-Span Steel Bridge with Fiber Reinforced Polymer Composite Deck Panels,” Transportation Research Record, 1770 (2001): 113-123

4) Reising, R.M.W. (2002), “Field Testing and Long-term Monitoring of a Five-Span Bridge with FRP Deck” Ph. D. Dissertation, University of Cincinnati.