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Load Testing of An FRP Bridge Deck On A Truss Bridge

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ABSTRACT

New York State has installed a Fiber Reinforced Polymer (FRP) bridge deck as an experimental project. The goal of the project was to improve the load rating of a 60-yr. old truss bridge located in Wellsburg, New York. The FRP deck weighs approximately 80-percent less than the deteriorated concrete bridge deck it replaced. Reducing the dead load increased the allowable live load capacity of the bridge without significant repair work to the existing superstructure, thus lengthening its service life. Load testing was conducted after installation of the FRP deck to study the conservativeness of the design, ascertain the assumptions made on composite action between the deck and the superstructure, and examine the effectiveness of joints in load transfer. The results indicate that the design was conservative. The design assumed no composite action between the deck and the superstructure, and the experimental data confirms that assumption. Peak strains under the test loads were only a very small fraction of the ultimate strength of the FRP deck. The study also shows that the joints are only partially-effective in load transfer between panels.

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I. INTRODUCTION

The nation's bridge infrastructure is deteriorating at an alarming rate. Due to the staggeringly high cost of repair and replacement, most transportation agencies are unable to cope with this trend. Nearly one-third of the nation's 580,000 bridges are classified deficient. Structural "deficiency" does not imply that a bridge is unsafe or likely to collapse. More than 29,000 bridges are classified as structurally deficient because of poor deck conditions or lack of load ratings (*1*). The Federal Highway Administration (FHWA) estimates that repair of deficient or obsolete bridges will cost more than 20 billion dollars (*2*).

The New York State Department of Transportation (NYSDOT) has similar problems with nearly 7585 (or 38.9 %) bridges identified as structurally or functionally deficient (*3*). Table 1 shows New York State bridge deficiency data sorted by ownership of the bridge. Nearly 2000 of these 7585 bridges were classified deficient due to poor deck conditions or weight restrictions. Much of the deck deterioration can be attributed to heavy application of road salts during the snowy winters experienced in the state.

Table 1. Current status of bridges in New York State (3).

Owner	Number of Bridges		
	Total	Deficient	% Deficient
State-owned	7,797	2,225	28.5
Locally-owned	8,898	3,902	43.8
Other	2,819	1,458	51.8
Total	19,514	7,585	38.9

The New York State Department of Transportation is constantly looking for new materials, methods, and technologies to cost-effectively replace old bridge decks and improve load ratings. Fiber reinforced polymer (FRP) composite systems are one such alternative under consideration. Fiber reinforced polymers are gaining popularity in the bridge community. These materials have high strength-to-weight ratios and excellent durability (corrosion resistance). They have a long record of use in Europe and Japan (*4*). New York recently began using and evaluating FRPs as viable alternatives for bridge deck repair, strengthening deteriorated components, removing load postings, and prolonging service life (*5-8*).

New York State has many old truss bridges with deteriorated superstructures, and these bridges are restricted to less than legal loads. Due to the nature of these bridges, replacement is often a cost-effective option. Since resources are limited, many of these bridges may not be replaced with new structures for several years. FRP decks seem to offer a cost-effective alternative to complete replacement since they are much lighter than conventional bridge decks. FRP decks not only replace deteriorated bridge decks, but also reduce the dead load. The allowable live load capacity increases with the dead load reduction and the rehabilitated bridges can carry legal loads without extensive repairs. The simple, modular nature of FRP deck construction is an additional benefit. Installation is relatively fast, reducing the inconvenience to traveling public.

New York State has installed a fully Fiber Reinforced Polymer (FRP) bridge deck on a truss bridge as an experimental project. The goal of the project was to improve the load rating of a 60-yr. old truss bridge located in Wellsburg, New York. The FRP deck weighs approximately 80-percent less than the deteriorated concrete bridge deck it replaced. Reducing the dead load allowed an increase to the allowable live load capacity of the bridge without significant repairs to the existing superstructure, thus lengthening its service life. Load testing was conducted after installation of the FRP deck to study the conservativeness of the design, ascertain the assumptions made on composite action between the deck and the superstructure, and examine the effectiveness of joints in load transfer. This report describes the testing and discusses the results.

II. BRIDGE REHABILITATION

This section briefly describes the old bridge structure and rehabilitation of the bridge with an FRP deck.

A. BRIDGE STRUCTURE

The bridge carrying State Route 367 over Bentley Creek in the village of Wellsburg, Chemung County, New York, was erected in 1940 (see Figure 1). It is a simply supported, single-span, inclined top chord, Warren steel truss structure with a concrete deck and asphalt wearing surface (*9*). The bridge is 42.7 m long, 7.3 m wide curb to curb, and has a skew of 27 degrees. The floor system consists of steel wide-flange floor-beams and stringers. A 1.85-m wide sidewalk is located outside the east truss. The substructure consists of concrete abutments and wing walls, supported by timber piles (*9*). The bridge carries two lanes of traffic, has an Average Daily Traffic flow (ADT) of 3248, and 7% of the ADT is truck traffic (*10*).

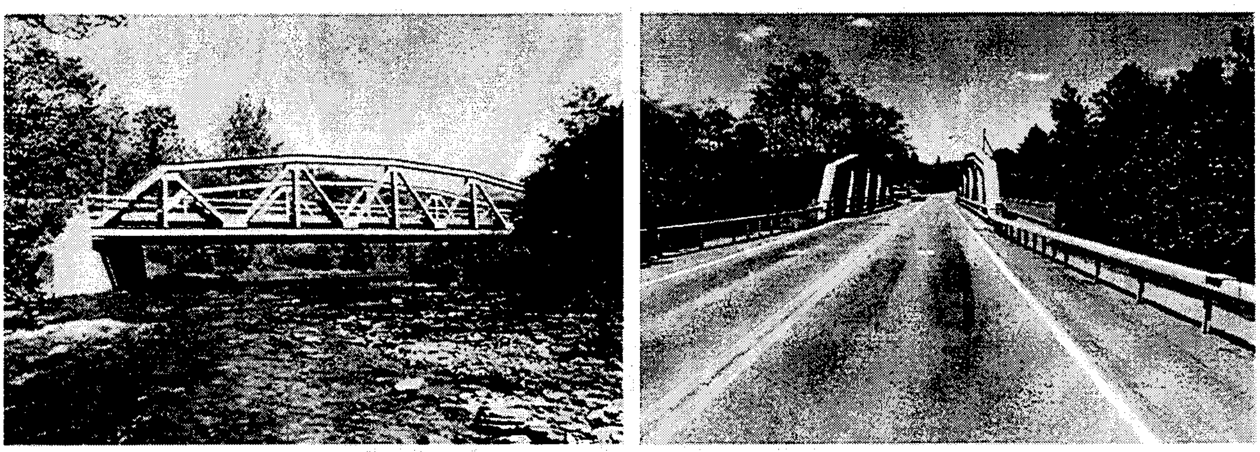

Figure 1. Views of Bentley Creek Bridge before rehabilitation.

Even though the Bentley Creek Bridge was previously rehabilitated in 1978, the bridge was weight restricted to 14 tons by the end of 1997 due to the results of an updated load rating. The new rating reflected additional dead load from asphalt overlays applied over time, steel corrosion on the trusses and floor system (see Figures 2-4), and the poor condition of the deck. The existing deck was 180-mm of concrete overlaid with a 170-mm asphalt wearing surface weighing 8.13 kPa. The inspection

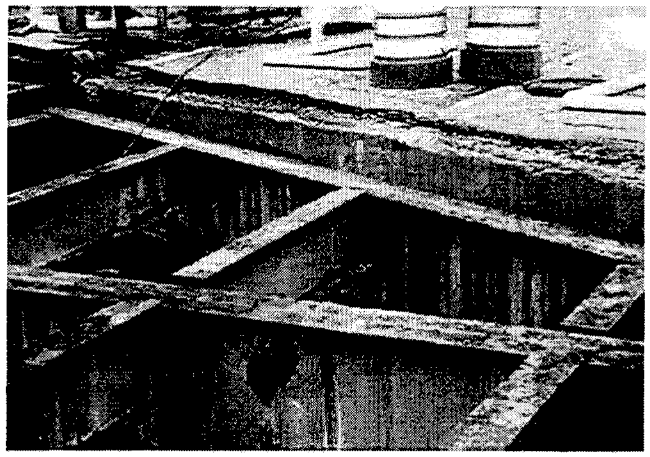

Figure 2. Corrosion of steel superstructure at the south abutment.

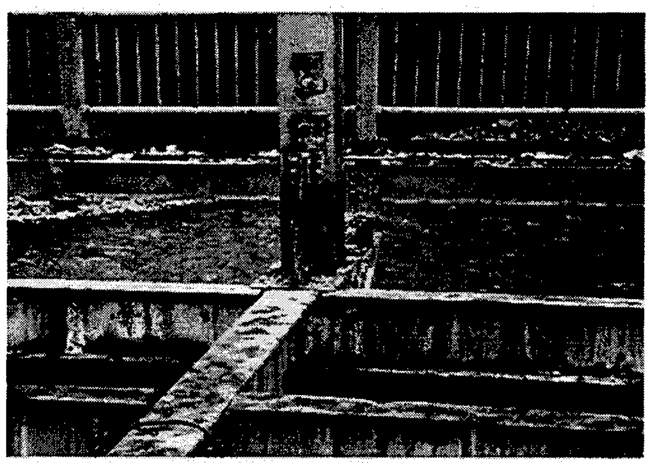

Figure 3. Corrosion at the floor-beam to truss connection.

Figure 4. Corrosion of floor-beams.

rating of the deck was 3, on a scale of 1 to 7 used in New York State (see Table 2), indicating serious deterioration. The steel trusses were found to be in relatively good condition even though the superstructure was built in 1940. The minor deterioration was attributed to leakage of water laden with de-icing salts. The load posting had a negative impact on local commerce and restoring the bridge to full service was imperative. Exploration of rehabilitation alternatives began in 1998.

Table 2. New York State Department of Transportation Condition Ratings (*11*).

Rating	Condition
1	Totally deteriorated
2	Used to shade between ratings of 1 and 3
3	Serious deterioration or not functioning as originally designed
4	Used to shade between ratings of 3 and 5
5	Minor deterioration and functioning as originally designed
6	Used to shade between ratings of 6 and 7
7	New condition

B. REHABILITATION

After careful consideration of the alternatives, replacing the existing deck with a lightweight FRP deck weighing 1.53 kPa combined with minimal repair of superstructure was selected (*10*). The rehabilitation not only prolongs the structure's service life, but also removes the load restrictions. This alternative was considered faster than other conventional alternatives and cost-effective compared to complete bridge replacement (*10*).

The rehabilitation included the following:

1. Removal of the concrete deck and the sidewalk,
2. Steel repairs (i.e. replacing rusted rivets with bolts, fish-plating areas of section loss, etc.),
3. Installation of temporary deck grating,
4. Cleaning and painting of steel superstructure,
5. Removal of temporary grating and installation of light-weight FRP deck (see Figure 5), and
6. Replacement of approach and bridge railings (see Figure 6).

The FRP composite panels were made by Hardcore Composites of Delaware. The deck is a cell core structure made of E-glass stitched fiber fabric wrapped around 150 mm x 300 mm x 350 mm isocycrinate foam blocks used as stay-in-place-forms. A vacuum assisted resin infusion process was used (*2*) with vinyl ester resin. The deck was designed for AASHTO MS23 live-load (*12*) using finite element analysis. Orthotropic in-plane properties were used in the analysis. Stresses in the composite materials were limited to 20% of their ultimate strength; and deflection was limited to span/800.

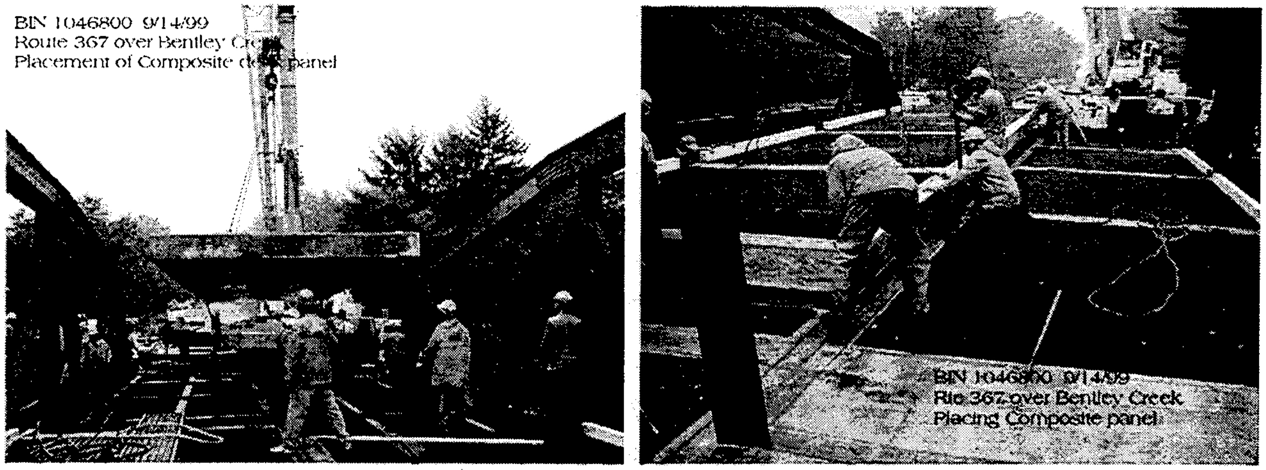

Figure 5. Composite deck panels being positioned with a crane.

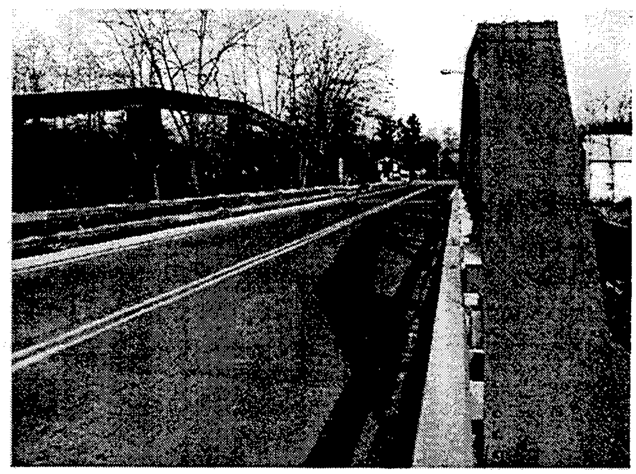

Figure 6. Bentley Creek bridge after rehabilitation.

The deck panels were designed to span between the floor-beams. The steel stringers were left in place to provide bracing to the structure, although they no longer function in carrying live load. A total of six FRP panels were used to replace the bridge deck. Each panel was 3.8 m wide and lengths varied from 12.8 m to 16.2 m. End panels were skewed 27 degrees. Bearing pads made of 6 mm thick neoprene pads were placed across the full length of the transverse floor-beams to provide uniform bearing between the structural steel and the FRP deck. A polymer concrete haunch was placed on top of the bearing pads to provided a cross slope to the bridge deck.

Seventy-five mm diameter holes were drilled through the top face skin and foam core of the deck panels. A twenty-five mm diameter hole was then drilled through a steel plate in the bottom face of the composite deck, haunch material and the top floor-beam flange. A structural bolt attached the deck to the superstructure. The drilled holes were then filled with a non-shrink grout (see Figure 7). For the design and load rating of Bentley Creek bridge, composite action was not assumed. However, the bolt connections were conservatively designed to withstand any composite action forces that might be present.

The panels were connected to each other using epoxy. The epoxy joints consist of a longitudinal joint that runs the entire length of the bridge and four transverse joints that each span one lane (see Figure 8). A 10-mm thick Transpo T-48 epoxy thin polymer overlay was used as the wearing surface of both the deck and sidewalk. Most of the wearing surface was applied to the panels during fabrication. Portions of the wearing surface covering panel joints and bolt lines were applied on-site. In June 2000, the entire wearing surface was removed because of adhesion problems stemming from inadequate surface preparation prior to installation. A new wearing surface, similar to the original, was put on the FRP deck.

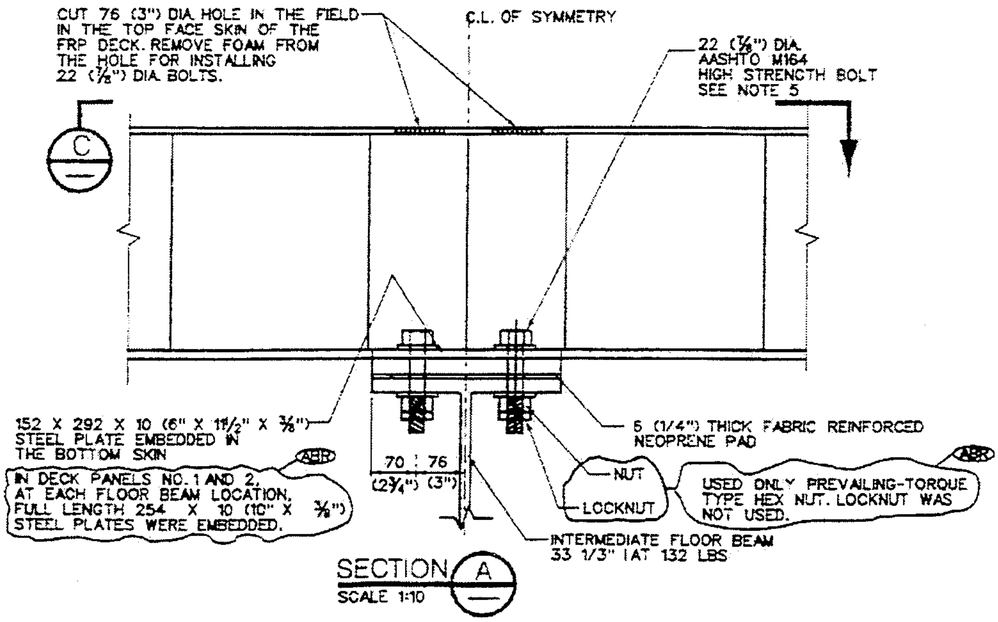

Figure 7. Bolt connection detail.

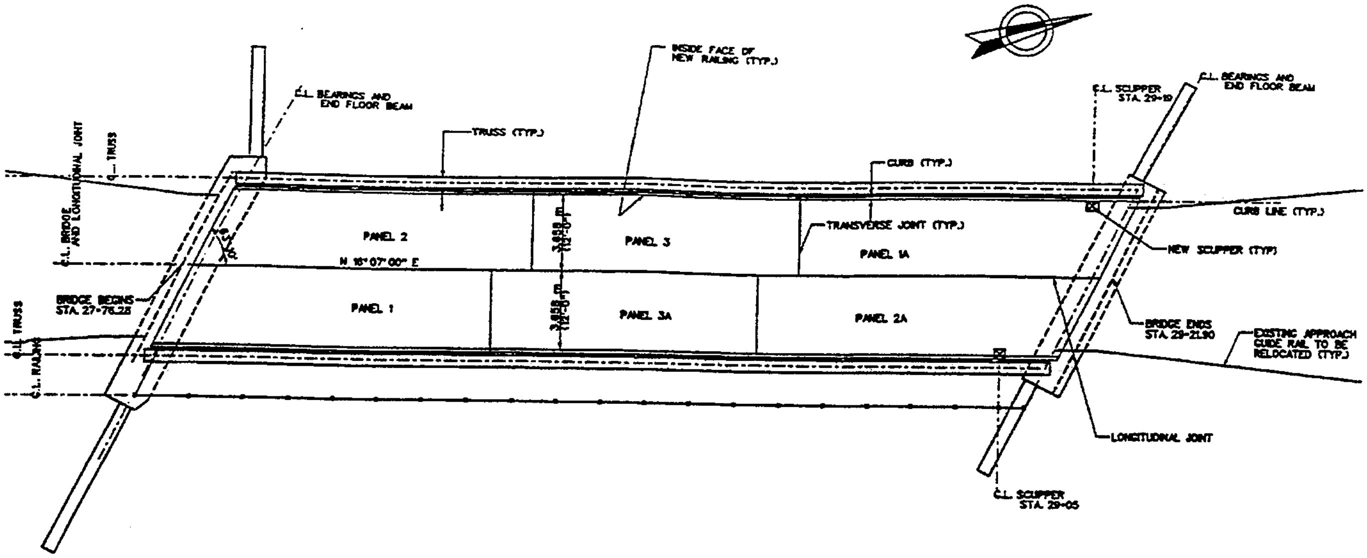

Figure 8. Composite deck plan.

III. LOAD-TESTING

FRP decks are relatively new to civil engineering applications and this was the first FRP deck installed on a state highway in the United States. There are no proven analysis procedures or design standards available. Thus, FRP deck behavior under live loads and long-term durability are not well understood by civil engineers. Several assumptions were made during the design of the FRP deck system which was designed very conservatively. Since the true extent of composite action was unknown, the connections (see Figure 7) were conservatively designed assuming full composite action forces between the deck and floor-beams.

An as-installed field evaluation through load testing and further analytical investigations were considered essential to ensure the safe and cost-effective use of FRP decks in future NYSDOT projects. A load-test with known truck weights was conducted to verify some of the design assumptions considered critical for future projects. A decision to develop a detailed calibrated finite element model of the bridge was made to further the understanding of the deck behavior under live loads and its failure mechanisms. This study will be published in another report. The experimental data collected during the load test will be used to calibrate the model for reliable simulations in a parametric study to investigate the effect of deck deterioration and minor damage to the composite panels.

A. FIELD TESTING OBJECTIVES

The Bentley Creek Bridge was load tested on November 17, 1999 with the following objectives:

- Determine if composite action exists between the FRP deck and the floor-beams
- Determine the effectiveness of the deck joints in transferring loads
- Verify the load rating of the deck
- Acquire strain data for developing a calibrated, detailed finite element model to further investigate the failure mechanisms associated with the bridge deck.

B. INSTRUMENTATION

Conventional, general purpose, uniaxial 350 Ω, self-temperature compensating, constantan foil-strain gages, manufactured by Measurements Group, Inc, were used to measure strains during the testing. The gages (Model EA-06-250AE-350) have an open-faced construction, with a 0.03 mm flexible polyimide film backing, a nominal gage factor of 2.075 and a -75 to 175^0 C temperature range for continuous use in static measurements. The strain gages were bonded to the steel and FRP deck with M-Bond 200 adhesive and then waterproofed by applying air-drying solvent-thinned polyurethane coating. Rubberized self-adhesive sheets (M-Coat FB-2 Butyl Rubber Sealant) were applied to weatherproof the instrumentation.

Strain gage locations were selected to meet the objectives of the testing as described in the previous section. A total of 18 strain gages were used, 6 placed on a steel floor-beam and 12 placed on the FRP deck (see Figures 9 and 10). The data was collected using SYSTEM 4000, a computerized data acquisition system manufactured by the Measurements Group, Inc.

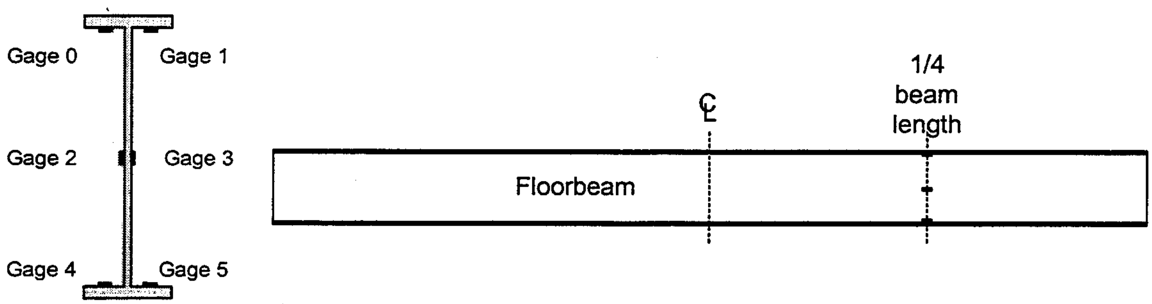

Figure 9. Location of strain gages mounted on the floor-beam.

C. TESTING

Three separate load cases were utilized. Two NYSDOT dump trucks (designated A and B) were used to load the bridge. Each fully loaded truck closely resembles a M-18 (H-20) AASHTO live-loading (*12*). Truck configuration and weights used in the testing are shown in Figure 11 and Table 3, respectively.

Table 3. Truck weights in kN.

Truck	Front Axle		Back Axle		Gross Weight
	Left	Right	Left	Right	
A*	44	41	64	65	214
B*	45	42	66	58	211

*Front axle tires are 2.30 m apart (center-to-center), rear axle tires are 2.50 m apart (center-to-center), and the distance between front and rear axles is 4.65 m (center-to-center).

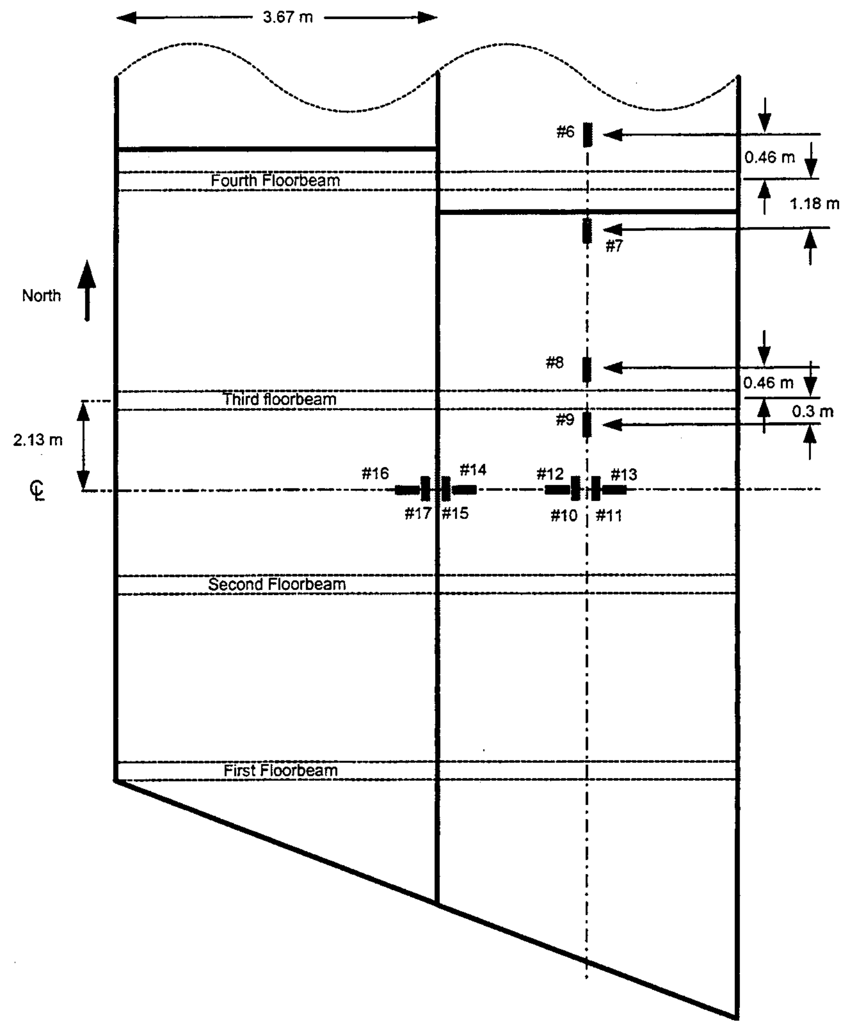

Figure 10. Locations of strain gages mounted on the FRP deck.

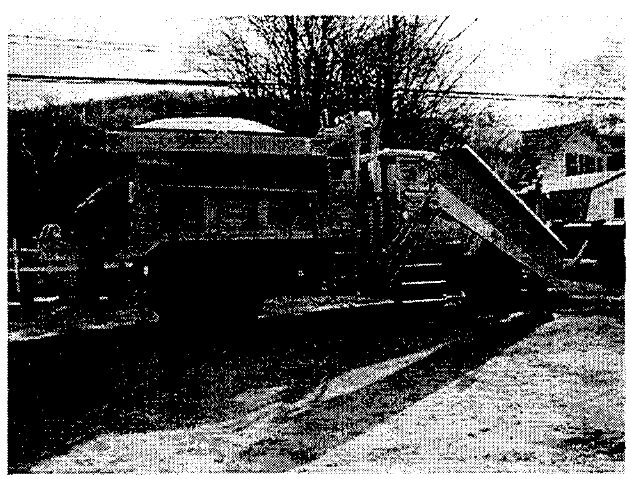

Figure 11. Typical loaded test-truck.

For the first load case, the rear axles of Trucks A and B were positioned in the instrumented bay, halfway between two floor-beams (see Figures 12 and 13). Each truck was moved on and off the bridge individually and data was recorded at each step. This sequence was repeated 3 times to ensure the consistency of the recorded data. Data from all three tests were then averaged to eliminate random noise. The second load case (see Figure 14) positioned both trucks back-to-back in the north lane. Again, each truck was moved on and off the bridge individually with data recorded at each step. Both of these load cases generated data for locating the neutral axis of a typical floor-beam as well as general strain distributions in the deck. Load Cases 1 and 2 also examine load transfer across the joints between FRP deck panels.

In the third load case, each truck was driven across the bridge -- only one truck was on the bridge at a time -- in the northbound lane at 5 km/hr. The data generated by the rolling loads will be used to create influence lines for calibration of a detailed finite element model.

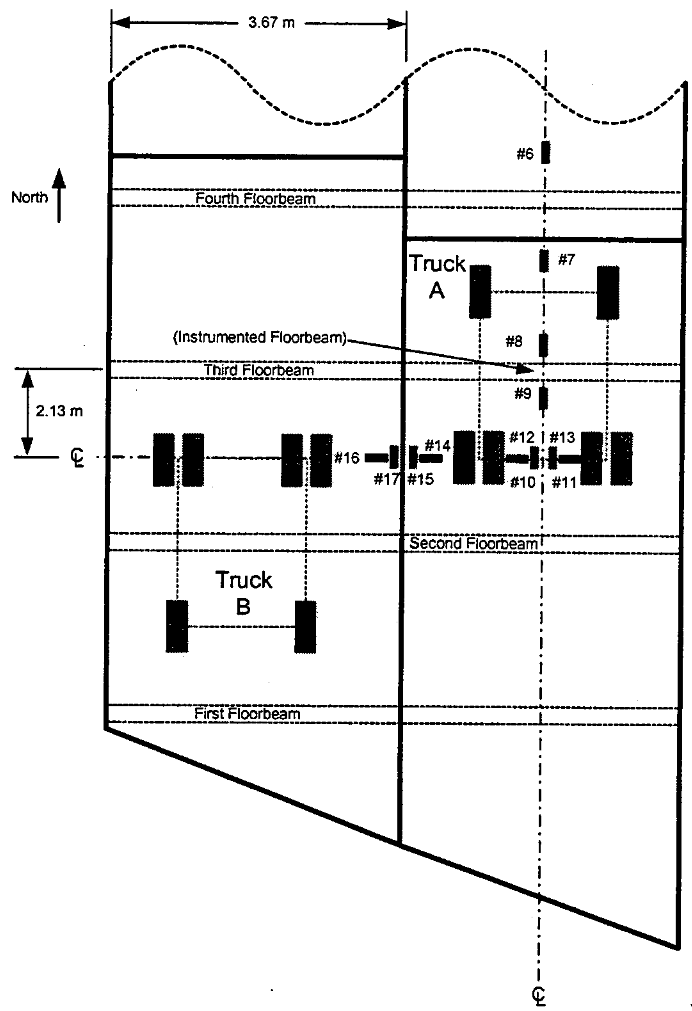

Figure 12. Truck positions for Load Case 1.

Figure 13. Trucks A and B in position - Load Case 1.

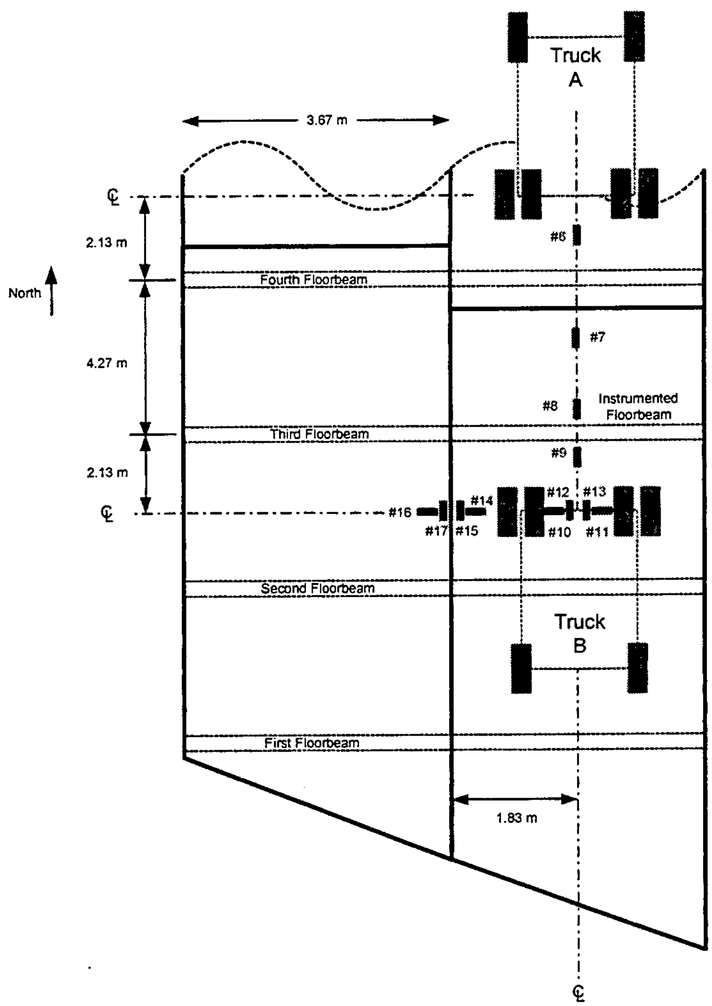

Figure 14. Truck positions for Load Case 2.

IV. LOAD TEST RESULTS

Data from the load test were analyzed to obtain strains in the deck and floor-beams supporting the deck. These results were used to investigate composite action between the FRP deck and floor-beams, the effectiveness of joints in load transfer across the panels, as well as the general behavior of the FRP bridge deck.

A. COMPOSITE ACTION

Strain gages 0, 1, 2, 3, 4, and 5 were mounted on a steel floor-beam supporting the FRP deck (see Figure 9) to determine neutral axis of the deck-floor-beam system. Gages 0 and 1 were mounted to the upper flange, gages 2 and 3 were mounted on the web at mid-height of the girder (at the neutral axis of the floor-beam), and gages 4 and 5 were mounted to the lower flange. If there is no composite action between the floor-beam and the deck, neutral axis of the deck-floor-beam system should coincide with the neutral axis of the floor-beam. In this situation, Gages 2 and 3 should read no flexural strains; and top flange gages (0 and 1) and bottom flange gages (4 and 5) should read strains of same magnitude with opposite signs (positive for tension, and negative for compression). Data collected from Gages 1, 3, and 5 during Load Cases 1 and 2 are shown in Table 4. The results show that the strains in bottom and top flanges are almost the same except for the sign (as expected) and represent a mirror image, with negligible strain at the center of the girder. The data indicates that the neutral axis of the girder is unchanged with the addition of the FRP deck and no composite action exists between the deck and the floor-beams. Redundant Gages 0, 2, and 4 confirmed this observation. The same behavior can be seen in the data collected during the passage of a truck rolling at 5 km/hr across the bridge (see Figure 15).

Table 4. Average readings of gages mounted on a floor-beam to examine neutral axis location.

	Average Strain ($\mu\varepsilon$)[1,2]	
Location	Test 1	Test 2
Top Flange (Gage 1)	-90	-43
Bottom Flange (Gage 5)	90	45
Middle of floor-beam (Gage 3)	3	-2

[1] Average of three cases, with two loaded dump trucks on the deck
[2] Negative strain indicates compression, positive strain indicates tension

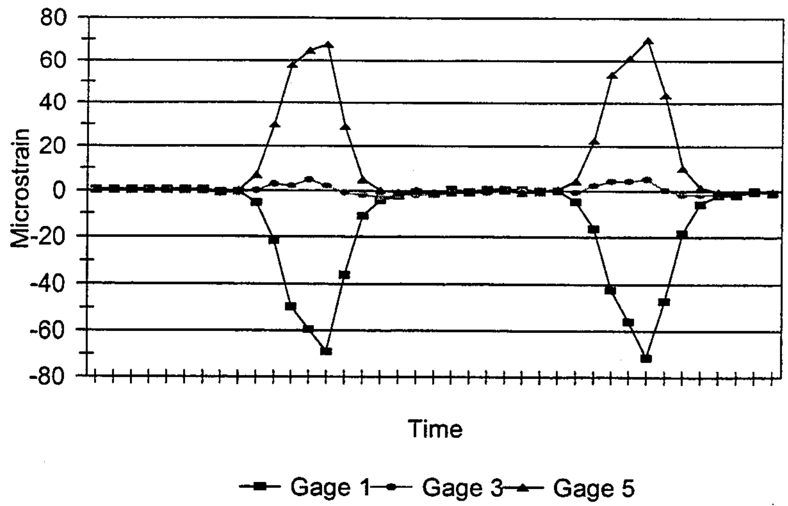

Figure 15. Gages on steel floor-beam during the 5km/hr rolling tests.

This validates the deck design, which assumed no composite action between the deck and the floor-beams, and the effectiveness of the connection system used to fasten the FRP deck to the floor-beams.

B. EFFECTIVENESS OF THE JOINTS

The deck panels were connected to each other with epoxy resin on the vertical faces as well as top and bottom FRP cover plates. There is no other mechanical shear-key mechanism. Hence to study the effectiveness of the joints, strain gages were installed on both sides of the longitudinal joint and loaded trucks were positioned in both lanes of the structure (Load Case 1 - see Figure 12), one truck at a time. The data from this load case is shown in Table 5. If the joint is transferring the loads effectively, strains recorded by the gages on either side of the longitudinal joints should be equal. With one truck located on deck panels on each side of the joint, (the case of Trucks A+B), both the gages read similar strains as expected since the trucks have near equal weights. When the load is only on one side of the joint, the readings changed noticeably. The data shows that the joint is transferring approximately 65 to 70% of the load. The same trend can be observed using the data obtained from semi-static load testing (see Figure 16). These results indicate that the longitudinal joint is transmitting loads from one deck segment to the other, but the load is not completely carried across the joint. Since the deck joints were new at the time of the testing, it is recommended that testing be repeated after a few years to determine if in-service loads and environmental conditions have degraded the joint effectiveness.

Table 5. Average strain data recorded with Gages 15 and 17 for joint analysis.

	Average Strain ($\mu\varepsilon$)[1]	
Loading	Gage 15	Gage 17
Truck A	75	49
Truck A + B	148	140
Truck B	75	93

[1] Average of three repeated tests

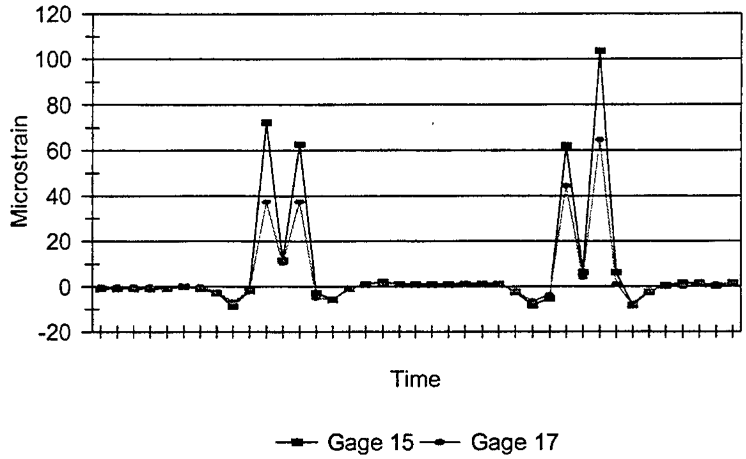

Figure 16. Gages both sides of the longitudinal joint during the 5km/hr rolling tests.

C. MAXIMUM STRAINS

The largest strain experienced by the floor-beam was about 95 $\mu\varepsilon$ and occurred in Test 1 with rear wheels of both trucks positioned on the deck midway between the floor-beams. The maximum strains recorded on the floor-beam during the truck traveling at 5 km/h in the northbound lane were about 70 $\mu\varepsilon$. The influence line generated during this test for bottom and top flange gages is shown in Figure 15. Note that these strains due to service loads are considerably less than the allowable steel strains.

The peak longitudinal strain observed by any of the longitudinal gages located on the FRP deck was 159 $\mu\varepsilon$ and occurred during Test 1 with both trucks on the bridge, one in each lane (see Figure 12). The gage (#17) location was in line with the rear axles of the trucks, close to the joint, halfway between the second and third floor-beams from the south abutment. The maximum transverse strain recorded was 90 $\mu\varepsilon$ at Gage 14 with only Truck B placed on the deck during Load Case 2.

Two types of laminates (QM6048 and Q9100) were used in the construction of the FRP deck (*13*). QM 6048 has the lower modulus of elasticity (18,479 MPa), ultimate tensile strength (310 MPa), and ultimate compressive strength (221 MPa). Using this modulus of elasticity, the maximum strains correspond to 2.9 MPa and 1.6 MPa longitudinal and transverse stresses respectively. These values show that the stresses caused by service loads are very small when compared to ultimate strength of the FRP decks. Thus, the deflection criteria, limiting maximum deflection to span/800, controlled the design.

Figures 15 and 16 show the strain distributions, under live load, in both the floor-beam and the bottom face of the deck. The data shows that the deck strains directly under wheel loads are a combination of global bending and local bending. The results show that the strains are very high under the wheel loads and rapidly decrease (when compared to floor-beam strains), indicating that local bending effects dominate the deck strains. The same observation was noted by other researchers during the load testing of FRP bridge decks (*14*) and should be considered when designing wearing surfaces for FRP decks.

D. LOAD RATING

The deck was designed for AASHTO MS 23 (HS25) live load by the manufacturer. The ratings submitted to the Department were based on a finite element model developed using STADD software (*13*). Ratings were calculated assuming the allowable operating stresses to be 75% of the ultimate strength, and allowable inventory stresses to be 75% of the operating stress levels. The reported governing operating and inventory load ratings of the deck under flexure were MS-300 (HS-330), and MS-226 (HS-247). Shear controlled the rating and the corresponding operating and inventory load ratings of the deck were MS-105 (HS 115) and MS-78 (HS-85) respectively.

The test data was used to verify these ratings in flexure. Note that access to deck panel webs was not available, thus shear stresses were not measured during the load testing. Load ratings for flexural stresses were calculated, assuming that the deck is simply supported on the floor-beams. According to the project design specifications, an allowable strain of 20% of the ultimate strength and an impact factor of 0.3 were used in the calculations. Data from loads applied in Load Case 1 (rear axles of both trucks in line with Gages 10 and 11) were used for the analysis. The operating and inventory load ratings based on flexure were calculated to be MS-292 (HS-318) and MS-219 (HS-239) respectively. These results agree closely with the ratings reported by the manufacturer.

E. DATA FOR FURTHER ANALYSIS

Composites are new to civil engineering and a majority of bridge engineers are not experienced with their use. FRP behavior under live loads, failure mechanisms, behavior of connections, and long-term durability to environment factors and degradation are not well understood. There are no proven analysis procedures or design standards available. Most FRP systems built were designed and fabricated very conservatively. The load rating section of this report clearly indicates this trend.

In order to study the behavior of this FRP deck under live loads, examine its failure modes, and study the influences of damage the deck might incur during its life cycle, a detailed finite element model is currently being developed. To calibrate the finite element model, several strain gages were mounted to obtain data on the FRP panel during the field testing. This data is presented graphically in Figures 17-18 and tabulated in Tables 6-7.

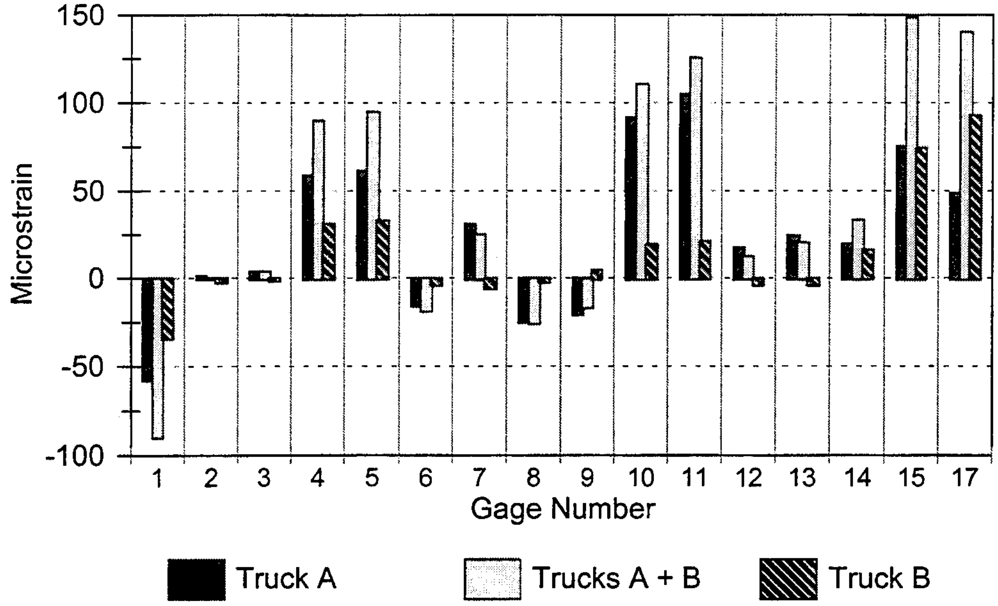

Figure 17. Average measured strains from Load Case 1.

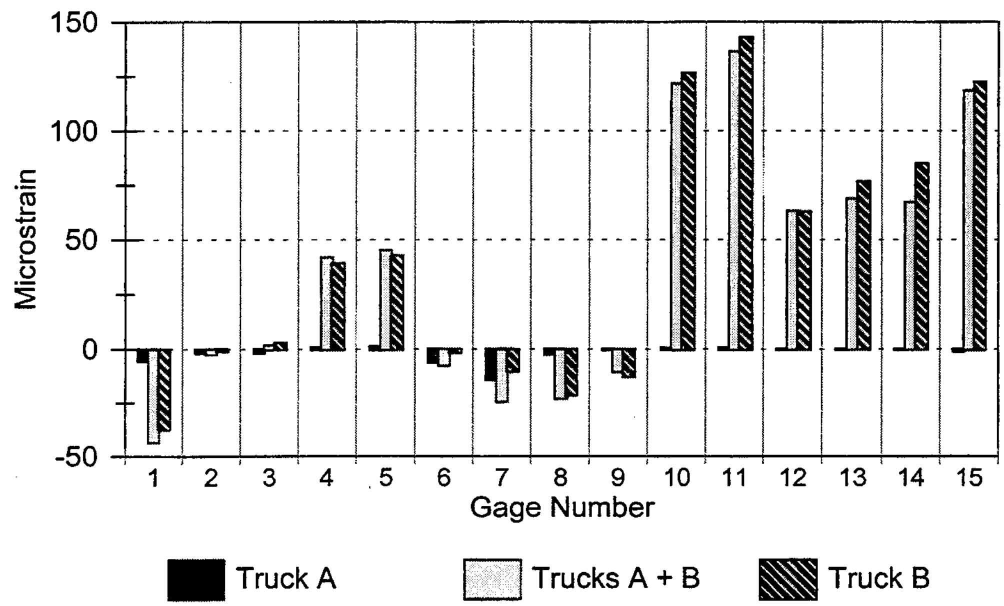

Figure 18. Average measured strains from Load Case 2.

Table 6. Averaged strain readings (με) for Load Case 1.

	Gage Number[1]															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	17
Truck A	-57	1	4	59	61	-15	31	-24	-20	92	105	17	25	20	75	49
Trucks A+B	-89	0	4	90	95	-18	25	-26	-16	111	126	12	20	33	148	140
Truck B	-33	-2	0	31	33	-3	-6	-2	4	19	21	-4	-3	16	75	93

[1] Gages 0 and 16 malfunctioned.

Table 7. Averaged strain readings (με) for Load Case 2.

	Gage Number[1,2]														
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Truck A	-5	-1	-1	1	1	-6	-14	-2	0	1	1	0	0	0	-1
Trucks A+B	-43	-2	2	42	45	-7	-24	-22	-10	122	136	63	69	67	118
Truck B	-37	-1	3	39	43	-1	-10	-21	-12	127	143	63	77	85	122

[1] Gages 0 and 16 malfunctioned.
[2] Gage 17 began to drift.

V. CONCLUSIONS

The first fiber reinforced polymer deck, installed on a 42.7-m truss bridge in New York State was load tested to verify conformance to specified strength criteria. The tests also provided a means to evaluate the assumptions made on composite action between the deck and the superstructure, and examine the effectiveness of joints in load transfer. The results indicate that the FRP deck was designed and fabricated conservatively. No composite action between the deck and the superstructure exists as assumed in the design. The study also showed that the joints are only partially effective in load transfer between different panels. A future load test should be considered to determine if the combination of in-service loads and environmental exposure weaken the joints. The test data indicates that localized bending effects may play a role in the strain distribution of FRP decks. These bending effects should be appropriately considered in the design of FRP bridge components such as wearing surfaces. Load test results will be used to calibrate a detailed finite element model for a parametric study to optimize the deck design and to further investigate the failure mechanisms.

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