Performance Evaluation of a Short-Span Bridge Built with FRP Reinforced Concrete Panels

by U. Deza and A. Nanni

Synopsis: This paper describes the evaluation of the in-service performance of a short-span bridge deck built with FRP reinforced concrete (RC) panels. The Walters Street Bridge consists of nine FRP-RC panels connected with shear keys. The multi-panel bridge deck was monitored for a period of four years by load testing the bridge deck with standard trucks and collecting deflection data. Experimentally derived factors such as stiffness degradation, and load fraction distribution between panels were computed from field deflections and compared with AASHTO provisions and results of an analytical study. The load fraction values, which assess the transverse load distribution, were consistent along the 4-year period. Load tests involving the use of two trucks at the same time were performed in the last year with the purpose of finding the most critical deflection under service load conditions, which was compared to allowable AASHTO live load deflections. Analytical deflections were calculated using ACI guidelines and structural analysis methods. The load tests, as well as the analytical results, revealed that the deflections were well within the recommended AASHTO values.

Keywords: FRP reinforced concrete panels; impact factor; live load deflection; load fraction distribution; multi-panel bridge deck; nondestructive load test; stiffness degradation

Ursula Deza is a doctoral student at the Civil Engineering Department of Iowa State University. She received her BS in Civil Engineering from the Pontifical Catholic University of Peru and her MS in Civil Engineering from the University of Missouri - Rolla. Her research interests include structural analysis, design and non-destructive load testing of bridges built with concrete, steel and composite materials.

Antonio Nanni (FACI) is the V&M Jones Professor of Civil Engineering at the University of Missouri-Rolla. He is the founding Chair of ACI Committee 440, Fiber-Reinforced Polymer Reinforcement, and is the current Chair of ACI Committee 437, Strength Evaluation of Existing Concrete Structures. He is an active member in technical committees of ASCE (Fellow), ASTM, and TMS.

INTRODUCTION

As a result of advanced deterioration of a pre-existing concrete bridge, the City of St. James, Missouri, decided to replace a deteriorated bridge deck with FRP precast concrete panels. The project, conducted by the City of Saint James in conjunction with the University of Missouri - Rolla (UMR), provided an opportunity to study the structural performance of the multi-panel bridge deck for a four year-period. Walters Street Bridge is comprised of a deck built with nine FRP-RC panels spanning in the direction of the vehicular traffic (Fig. 1). Since its construction, the bridge was subjected to yearly non-destructive load tests for four years. The main objectives of this investigation were: (a) to examine the deflection data in service which could be correlated to allowable deflections, (b) to estimate if there was any stiffness degradation that can indicate distress in the deck, and (c) to compute the load fraction between panels based on experimental data.

DESCRIPTION OF THE BRIDGE DECK

The bridge deck, built in 2001, consists of nine-FRP reinforced precast concrete panels, 2-foot 10-inch (*864-mm*) wide, 24-foot (*7315-mm*) long and 1-foot (*305-mm*) deep (1). The panels are interconnected by shear keys consisting of steel angles embedded on each panel side, and filled with grout. The angles are welded to ½-in. (*12.7-mm*) diameter smooth steel bars to form the connection. Between panels and concrete abutments, there are elastomeric bearing pads. The panels are anchored to the abutments by 1-in. (*25.4-mm*) diameter bolts, embedded 24 in. (*609 mm*) in the abutment. A two-part epoxy, placed in pre-drilled holes, was used to anchor the bolts. The nuts used to tighten the bolts were accommodated in a recessed section, which in turn was filled with a non-shrink grout (Fig. 2). The deck has thrie-beam guardrails along both sides. The deck top surface was broom-finished, and the edges were rounded to ¾-in. (*19 mm*) radius.

The longitudinal flexural reinforcement is arranged in two layers. The top consists of four ½-in. (*12.7-mm*) glass FRP (GFRP) bars used to keep the stirrups in place during construction. The bottom reinforcement, primary reinforcement, consists of twelve bundles of three 3/8-in. (*9.52-mm*) diameter carbon FRP (CFRP) bars. Pairs of 3/8-in. (*9.52-mm*) diameter stirrups were distributed ranging from 5 in. (*127.5 mm*) near the supports, and 12 in (*304.8mm*) at mid span. For the CFRP bars, the tensile strength was 270 ksi (*1900 MPa*) and the tensile elastic modulus was 15200 ksi (*104.8 GPa*). For the concrete, the compressive design strength was 8 ksi (*55.2 MPa*); however, compressive tests obtained from core samples obtained in the field revealed that the actual strength was 4 ksi (*27.6 MPa*). The deck was designed according to ACI 440 guidelines for concrete reinforced with FRP bars (2) to carry a standard HS15-44 truck loading. The design load fraction (LF), that represents a portion of a wheel line carried by an individual panel, was equal to 0.49 according to AASHTO provisions (3).

LOAD TESTING METHODOLOGY

The load test equipment used for the field evaluation consisted of a self-contained data acquisition unit with the capacity of monitoring 12 channels of deflections. The instrumentation utilized during the load tests included twelve direct current variable transformer (DCVT) transducers, installed under the bridge deck to monitor the deflection of the panels. The DCVTs were located at six locations underneath mid-span panels as well as six locations near the abutments (Fig. 3). The structural performance was monitored by load tests for a period of four years. A loaded tandem-axle truck with capacity of 25 tons (*22.7 tons*) was used every year.

A one-lane loading test, referred to as One-Truck Loading hereafter, was performed under static load conditions (1). The test protocol consisted of seven passes with five stops per pass (Fig. 4). The truck crawled at approximately 5 mph (*8.0 km/h*) and stopped at each position. Each stop lasted approximately 3 minutes, to allow for stabilization of the DCVTs readings.

In addition to the one-truck loading test, two trucks, in two different loading patterns were used in 2004 (4). The purpose of this test configuration was to exert higher deflections to the bridge and to compare the deflections to the allowable live deflection limitation of 1/800 provided by AASHTO (3). The first loading pattern consisted of two trucks placed on one lane with the rear axles back to back, hereafter referred to as Two-Truck One-Lane Loading (Fig. 5.a). This test was conducted only at the third stop (most critical condition) of Passes 1, 4 and 7 (Fig. 4). The second loading pattern consisted of placing the same two trucks in two lanes with the rear axles acting along the same line (side by side). Hereafter, this load configuration is referred to as Two-Truck Two-Lane Loading (Fig. 5.b). This test configuration intended to represent the most critical loading condition; thus, two trucks were placed in two lanes at the third stop (where maximum deflection was expected to occur) of Passes 1 and 7.

TEST RESULTS

The complete set of load vs. deflection curves for the 2001 Test are reported by Stone (1), the 2002 Test by Nanni (5), and the 2003 and 2004 Test by Deza (4). The collection of deflection readings of the deck due to the static truck loads was used to assess the deck response over time, determining the load fraction on the panels and validating analytical models. Herein, the presented data corresponds to the third stop Passes 1 and 4 where the maximum responses were obtained. It should be noted that only for the One-Truck One-Lane Loading Test, since the truck load was not the same every year, the deflections were normalized using the deck compliance defined as Δ/P (1*μin/kip = 5.7 μmm/kN in SI units*).

Based on the results of the load tests, for the One-Truck One-Lane Loading (Fig. 6), it can be concluded that the deformed shapes obtained are very similar after the 2001 Test, without significant loss of stiffness. The FRP-RC bridge deck maximum normalized deflection varied between 2847 and 3486 μin/kip (*16250 and 19898 μmm/kN*) for the most critical panel.

For the Two-Truck One-Lane Loading Test, the test results, presented in Fig. 7.a and b, were compared to the third stop of the One-Truck One-Lane Loading Test. After each test, the deflection readings for the unloaded structure did not show any residual deflection; which indicates that there was no distress caused to the deck. The largest deflection occurred for Pass 1 (*0.179 in = 4.55 mm*) being 30% larger than the deflection recorded when one truck was used (*0.138 in = 3.51 mm*) (Table 1).

For the Two-Truck Two-Lane Loading (Fig. 8), the largest deflection obtained in the field was 0.217 in. (*5.51 mm*), located under Panel 4. This deflection represents 73% of the allowable live deflection limit, 0.345 in. (*8.46 mm*) (L/800). When analyzing the same loading considering load superposition (sum of deflections corresponding to One-Truck One-Lane Loading in Pass 1 and Pass 7), the deflection computed under Panel 5 was 0.191 in. (*4.85 mm*). The difference between the field and superposed deflections is about 13.5%, which may be explained by the reduced moment of inertia in cracked panels, leading to a larger load test deflections. By comparing the One-Truck One-Lane Loading deflection of 0.135 in. (*3.43 mm*) with the Two-Truck Two-Lane Loading of 0.217in. (*5.51 mm*), the increment is about 60%.

The LF values were computed for each year from the mid span deflections. Previous investigations have shown that load fractions calculated from strain data and deflection data are nearly identical (6). Thereby, the LF for each panel was computed using the expression shown below:

$$LF_i = \alpha \cdot \frac{\Delta_i}{\sum_{j=1}^{n} \Delta_j} \qquad \text{Eq. (1)}$$

Where:

LF_i:	Load fraction computed for the i[th] panel
α:	2 (for one-lane LF); 4 (for two-lane LF)
Δ_i:	Load Test deflection of the i[th] panel
Δ_j:	Load Test deflection of the j[th] panel
n:	Number of panels in the deck

Table 2 shows that the values obtained from the tests are consistent, being 0.34 for One-Lane Loading (one or two trucks) and 0.48 for Two-Lane Loading. In both cases the deflections were less than LF of 0.49 calculated by AASHTO provisions. Using LF equal to 0.34 obtained from the load tests, using Eq. 1, the shear strength of the panels obtained was adequate (7). These results provide some assurance that the deck can support unexpected overloading.

It should be also noted that cracking on the deck soffit was not reported during the 2001 Test. The cracks were detected after performing the 2002 Test, and were located at the central third of the span. In the 2003 Test, a similar crack pattern was observed but no crack widths were measured. During the 2004 Test no significant changes in the cracking pattern were observed. The measured crack widths ranged from 0.003 in. (*0.076 mm*) to approximately 0.010 in. (*0.025 mm*) near the mid span. The crack widths were smaller than the allowable width of 0.020 in. (*0.050 mm*) specified by ACI 440.1R-03 (1). The existing cracking in the FRP-RC bridge deck did not dramatically affect the live load deflections for the last three years. Reasonable load fraction values were found from the field test results; which confirmed that the connections between panels (shear keys and grout) are adequate. The influence of cracking was observed in the load fraction distribution results (Fig. 9). Basically, when the panel exhibits more cracking, the load fraction decreases; and the difference of load is then carried by adjacent panels.

ANALYTICAL STUDY

Finite Element Method

Using a structural analysis software (SAP 2000), a finite element model (FEM) was developed to better understand the behavior of the bridge deck. The model intended to represent the behavior of the deck and the connections between the panels. Based on the material properties, original design considerations and construction procedures, the model was used to correlate the field and analytical deflections. The effect of dead load was not considered since the objective was to estimate the live load deflections. Also, no stiffening effect provided by the guardrails was considered. The truck load used in the model corresponded to the 2001 Test, with P1 = 16.28 kips (*72.4 kN*) for the middle axle, and P2 = 16.62 kips (*74 kN*) for the rear axle.

The bridge deck was modeled as a thick plate shell with a depth of 12 in. (*305 mm*), considering the gross cross section. The dimensions were 24 ft (*7315 mm*) long and 25 ft (*7620 mm*) width. The slab mesh allowed placing the wheels in similar locations to the actual field locations. The design length was 23 ft (*7010 mm*). Both support ends were restrained in three principal axes to simulate the effect of the anchors installed at the bridge abutments. The potential contribution of rotational spring stiffness provided by the anchors is small, and was neglected in the model. The concrete compressive strength was 4 ksi (*27.6 MPa*) (2). The modulus of elasticity was considered as 3254 ksi (*22.44 GPa*) and the Poisson's ratio was 0.18. The model considered the most critical case loading, which corresponds to Pass 1, Stop 3. During the 2001 Test, the concrete was considered to behave as linear elastic material (no cracking), since the bridge was tested after a month of construction completion.

The FRP-RC bridge deck was modeled in two different ways. Model 1 intended to idealize the multi-panel bridge deck as a solid slab, representing the ideal structural performance. The largest analytical deflection obtained was 0.135 in. (*3.249 mm*) measured at the same position as done in the field for Panel 1. Model 2 considered nine identical panels, 2 ft-10 in. (*864 mm*) wide, laterally interconnected by shear connectors. The shear connectors represented the shear keys filled with non-shrink grout to allow only transference of shear forces. The maximum analytical deflection was 0.153 in. (*3.89mm*), a value larger than the test results (*0.094 in = 2.388 mm*) (Fig. 10).

The analytical deflections obtained from both models were larger than the field values found in the 2001 Test. Thus, Fig. 11 shows the four-year period tests compared with the Model 2 results. Apparently, there is a combined action of the stiffness provided by the uncracked deck, the grouted shear keys and the bolts in the support that restrains the rotation. Since those parameters are unknown, the model was not able to accurately predict the deflection for the studied load configurations.

ACI Approach

Deflections according to ACI 440 guidelines were computed to predict the deflection for FRP-RC flexural members under service loads. The purpose was to estimate the deflections of a simply-supported single panel using load fraction values obtained from AASHTO Standards (3). The analytical deflections were then compared with the field deflections. The panel deflection was computed by using the relation of a simply supported beam loaded with the test truck. The effective moment of inertia of the section was calculated using the modified Branson's equation in 8.12-a (ACI 440-1R-03, 2003):

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \cdot \beta_d \cdot I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] \cdot I_{cr} \le I_g \qquad \text{Eq. (2)}$$

Where:

I_e	Effective moment of inertia, in^4 (*mm^4*)
M_{cr}	Cracking moment, lb-in
M_a	Maximum moment in a member at stage deflection is computed, lb-in (*N-mm*) at the 3rd stop
I_g	Gross moment of inertia, in^4 (*mm^4*)
b_d	Reduction coefficient used in calculating deflection (Equation 8.12-b, ACI 400-1R-03, 2003) is defined as:

$$\beta_d = \alpha_b \cdot \left(\frac{E_f}{E_s} + 1\right) \qquad \text{Eq. (3)}$$

Where:

α_b:	Bond dependent coefficient assumed as 1 for service conditions
E_f	Modulus of Elasticity of the CFRP bar: 15,200 ksi (*105 GPa*)
E_s	Modulus of Elasticity of the Steel bar: 29,000 ksi (*200 GPa*)

Table 3 summarizes the deflection values computed using LF_{AASHTO}=0.49 and LF_{test} obtained from the load tests, which ranged from 0.33 to 0.35 for the One-Truck One-Lane Loading Test for the four-year study, and 0.34 for Two-Truck One-Lane Loading and 0.48 for Two-Truck Two-Lane Loading for the 2004 Test. Observing the results tabulated in Table 3, the ACI deflection computed using LF_{AASHTO}, provide conservative values for each loading configuration. The ACI deflection using LF obtained from the field test provides more realistic values. In addition, the deflection due to the standard HS15-44 truck was estimated using an LF_{test} average value equal to 0.34, which provided a deflection value of 0.112 in. (*2.84 mm*) (L/2464). This value is smaller than the allowable live load deflection limitation of 0.345 in. (*8.76 mm*) (L/800). For the standard HS20-44 and HS25-44 trucks, the computed deflections were also below the live load deflection limit.

Double Integration Method

Deflections were also computed by double-integrating the Moment-Curvature Diagram (M-ϕ Diagram), which describes the flexural behavior of an FRP-RC section. All the knock-down factors for the CFRP and GFRP bars, used for design, were considered to be equal to 1.0, since the objective was to evaluate the actual behavior of the structure. To model the concrete under compressive stresses the well-known approach proposed by Todeschini was used (8). To compute the Moment-Curvature Diagram, the panel was assumed to have simply supported conditions with concentrated loads representing the wheel loads, which were multiplied by the corresponding LF.

The results show that the deflection corresponding to the LF_{AASHTO} of 0.49 is conservative compared to the field test results (Table 4), being approximately three times larger for the deflection obtained in the One-Truck One-Lane Loading. When using LF_{test} values, the results are more similar, with a difference ranging between 5% and 15% for the 2002 to 2004 Tests. In the 2001 Test, this difference was around 50%, which can be justified by the non-cracked condition of the slab. In the case of Two-Truck One-Lane and Two-Lane Loading, the double integration approach provided a deflection value two times larger than the value obtained in the field.

Additionally, the expected deflection for the standard HS15-44, HS20-44 and HS25-44 trucks was computed using LF_{AASHTO} and $LFtest_{-avg}$. From the results, note that for HS25-44, the L/800 allowable deflection is exceeded when using LF of 0.49. The deflection due to a standard HS20-44 truck was 0.301 in. (*8.41 mm*), which results in L/917 that is smaller than the design limitation of L/800. By refining the computation using $LF_{test\text{-}avg}$ equal to 0.34, the expected deflection for the standard HS25-44 was estimated as 0.203 in. (*5.16 mm*), which results in L/1360 that represents 60% of the allowable live load deflection limitation of L/800.

As a summary, deflections calculated by the ACI and double integration approaches, using LF_{AASHTO} = 0.49, exhibited similar deflection values. When using the different LF values obtained from the load tests, for the One-Truck One-Lane Loading test during the four-year study, and using the double integration approach, the obtained deflections had values closer to the field deflections. However, for the double loading, the deflection values are at least 75% larger than the field results. This difference may be explained by the unknown behavior of restraint at the support region.

CONCLUSIONS

Based on the results of the load tests, for the One-Truck One-Lane Loading the following can be concluded:

- From the 2001 Test results, it is apparent that the deck was not cracked at the time of the test.
- The deflections are consistent after the 2001 Test, without significant loss of stiffness.
- Reasonable LF values were found using field deflections as compared to LF values obtained using the AASHTO standard, which confirms to some extent that the connection between panels (shear keys and grout) are adequate.
- The existing cracking does not dramatically affect the results for the last three years. The largest crack width found in the field was below the allowable value recommended by ACI 440 1R-03.
- The influence of cracking was observed in the load fraction results. When the panel has more cracks, the load fraction decreases. The load difference is then carried by the adjacent panels.

From the results of the Two-Truck One-Lane and Two-Truck Two-Lane Loading Tests, the following can be concluded:

- The live load deflection were 52% and 60% of the allowable deflection limit L/800 for the most critical of One-Lane Loading and Two-Lane Loading, respectively.
- The results provide some assurance that the deck can support unexpected overloading.
- Using the LF values obtained from the tests, the shear strength of the panels was adequate.

From the results of the analytical study, it can be concluded that:

- In the first year, the structure was stiffer than the original design assumptions; therefore, may not have been cracked at the time of the load test.
- When introducing AASHTO LF values in the model, the prediction of deflections using ACI methods is about three times larger than the field results. However, when using the LF values obtained from test data, the prediction of deflections ranges from 25% to 40%.
- When using the double integration method and LF values obtained from the load tests, the analytical and field deflections have similar values.

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Table 1 - Comparison of the Maximum Deflections for Two Trucks

Year Test	Pass 1 (in) [mm]	Pass 4 (in) [mm]
One Truck: Truck 1	0.138 [3.51]	0.107 [2.72]
Two Trucks: Truck 1 & 2	0.179 [4.55]	0.151 [3.84]
Increment (%)	29.7%	41.1%

Table 2 - Load Fraction from Experimental Deflections

Test	Pass 1	Pass 4	LF	Difference %
One Lane $\alpha = 2$				
2001 – One-Truck	0.35	0.29	0.35	--
2002 – One-Truck	0.32	0.30	0.32	9%
2003 – One-Truck	0.34	0.29	0.34	3%
2004 – One-Truck	0.33	0.28	0.33	6%
2004 – Two-Truck	0.34	0.30	0.34	3%
Two lane $\alpha = 4$				
2004 – 2-Truck	0.48		0.48	

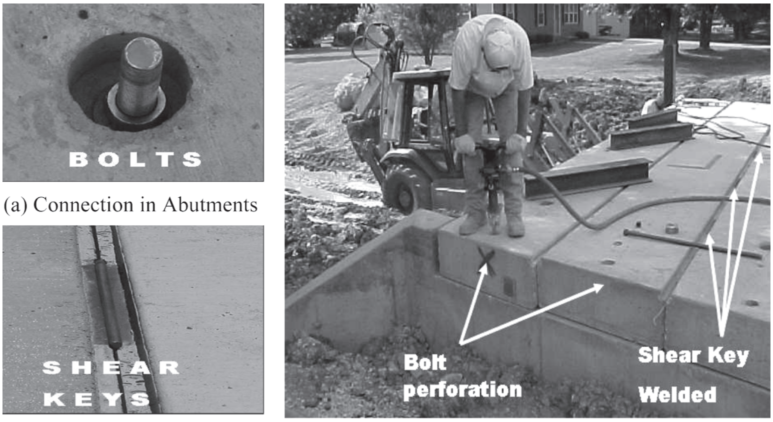
Table 3 - Deflections by ACI approach

Load Truck	Truck Position	Δ (in) /[mm] $LF_{AASHTO} = 0.49$	LF_{test}	Δ (in) /[mm] using LF_{test}	Δ_{test} (in) /[mm]
2001	1-Truck 1-Lane	0.255 [6.45]	0.35	0.156 [3.96]	0.089 [2.26]
2002	1-Truck 1-Lane	0.409 [10.38]	0.32	0.167 [4.24]	0.134 [3.40]
2003	1-Truck 1-Lane	0.388 [9.85]	0.34	0.174 [4.42]	0.125 [3.81]
2004	1-Truck 1-Lane	0.413 [10.49]	0.33	0.173 [4.39]	0.135 [3.43]
2004	2-Truck 1-Lane	0.893 [22.68]	0.34	0.335 [8.51]	0.180 [4.57]
2004	2-Truck 2-Lane	0.413 [10.49]	0.48	0.394 [10.01]	0.217 [5.51]
HS15-44	1-Truck 1-Lane	0.162 [4.155]	0.34	0.112 [2.84]	-- --
HS20-44	1-Truck 1-Lane	0.363 [9.22]	0.34	0.150 [3.81]	-- --
HS25-44	1-Truck 1-Lane	0.632 [16.05]	0.34	0.241 [6.12]	-- --

Table 4 - Deflections by Double Integration Approach

Load Truck	Truck Position	Δ (in) /[mm] $LF_{AASHTO} = 0.49$	LF_{test}	Δ (in) /[mm] using LF_{test}	Δ_{test} (in) /[mm]
2001	1-Truck 1-Lane	0.251 [6.38]	0.35	0.131 [3.33]	0.089 [2.26]
2002	1-Truck 1-Lane	0.400 [10.16]	0.32	0.140 [3.56]	0.134 [3.40]
2003	1-Truck 1-Lane	0.380 [9.65]	0.34	0.146 [3.71]	0.125 [3.18]
2004	1-Truck 1-Lane	0.403 [10.24]	0.33	0.144 [3.66]	0.135 [3.43]
2004	2-Truck 1-Lane	0.886 [22.5]	0.34	0.378 [9.60]	0.180 [4.57]
2004	2-Truck 2-Lane	0.403 [10.24]	0.48	0.381 [9.68]	0.217 [5.51]
HS15	1-Truck 1-Lane	0.145 [3.68]	0.34	0.098 [2.49]	-- --
HS20	1-Truck 1-Lane	0.301 [7.65]	0.34	0.131 [3.33]	-- --
HS25	1-Truck 1-Lane	0.531 [13.33]	0.34	0.203 [1.10]	-- --

Fig. 1 Walters Street Bridge – May 2004



(a) Connection in Abutments

(b) Connection between Panels

(c) Bolt Location

Fig. 2 Connections of FRP-RC Deck

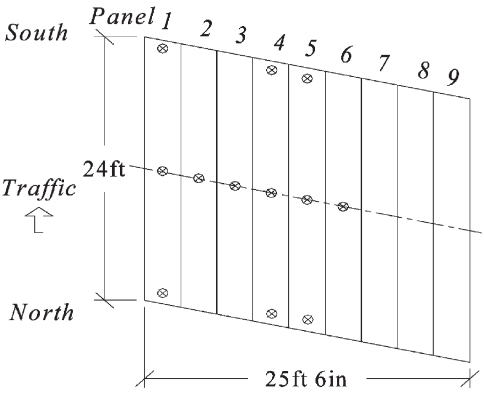
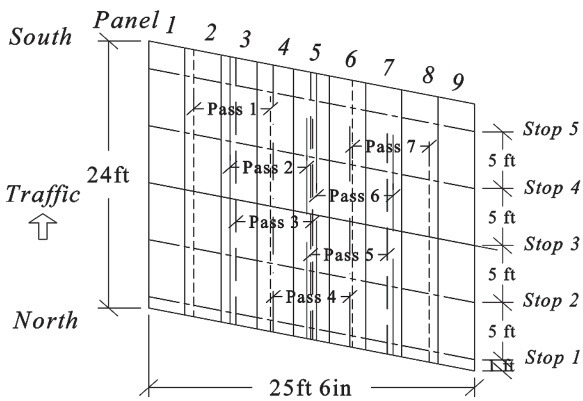


Fig. 3 Instrumentation Layout (US Units; *1ft = 305 mm, 1in = 25.4mm*)

a) Test Truck



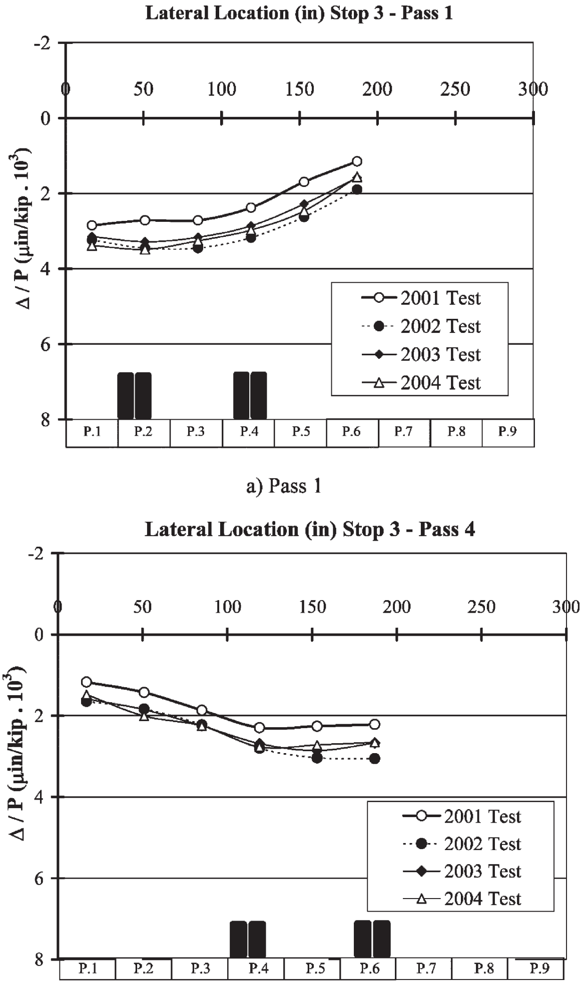
b) Truck Path

Fig. 4 One-Truck One-Lane Loading (US Units; *1ft = 305 mm, 1in = 25.4mm*)

a) One-Lane Loading

b) Two-Lane Loading

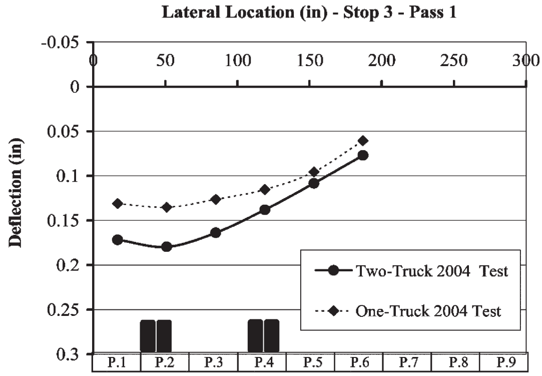
Fig. 5 Loading with Two Dump Trucks



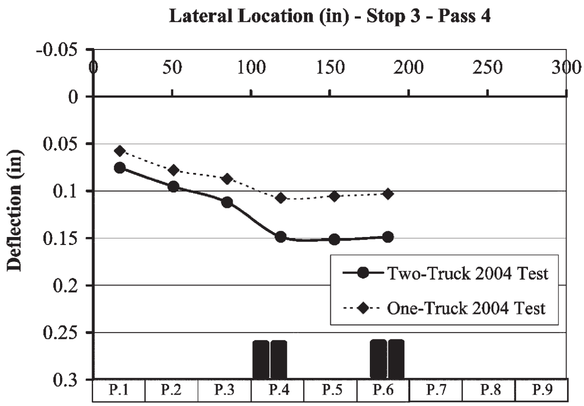
a) Pass 1

b) Pass 4

Fig. 6 Maximum Deformed Shape – One-Truck One-Lane Loading Tests (US Units; *1in = 25.4mm, 1min/kip= 5.7 mmm/kN*)



(a) Pass 1



(b) Pass 4

Fig. 7 One-Lane Loading Test: Two-Truck vs. One-Truck – Pass 1 (US Units; *1in = 25.4mm*)

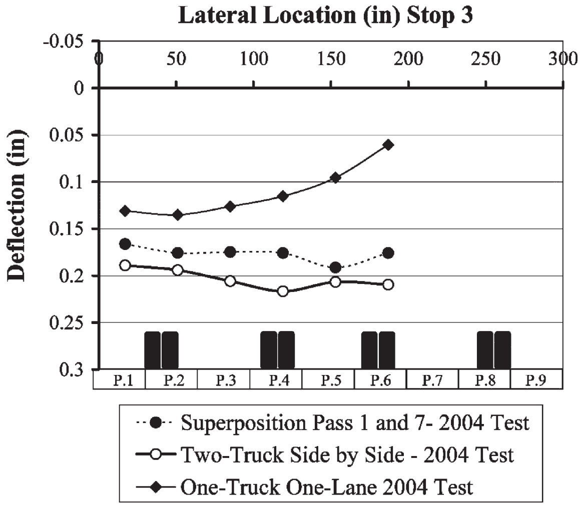


Fig. 8 Two-Lane Loading Test: Two-Truck vs. One-Truck – Pass 1 (US Units; 1in = 25.4mm)

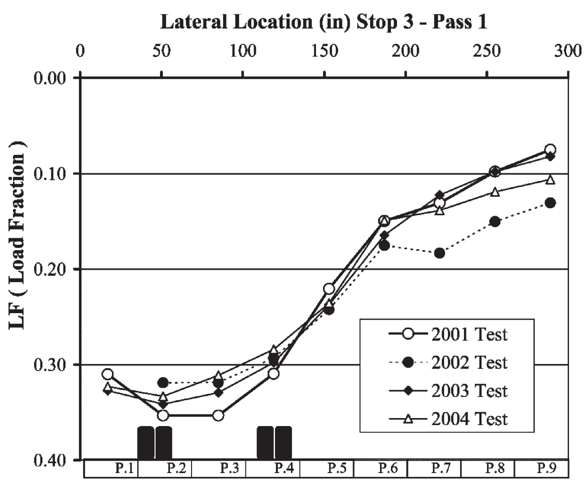


Fig. 9 Load Fraction Distribution along Four-Year Period Test

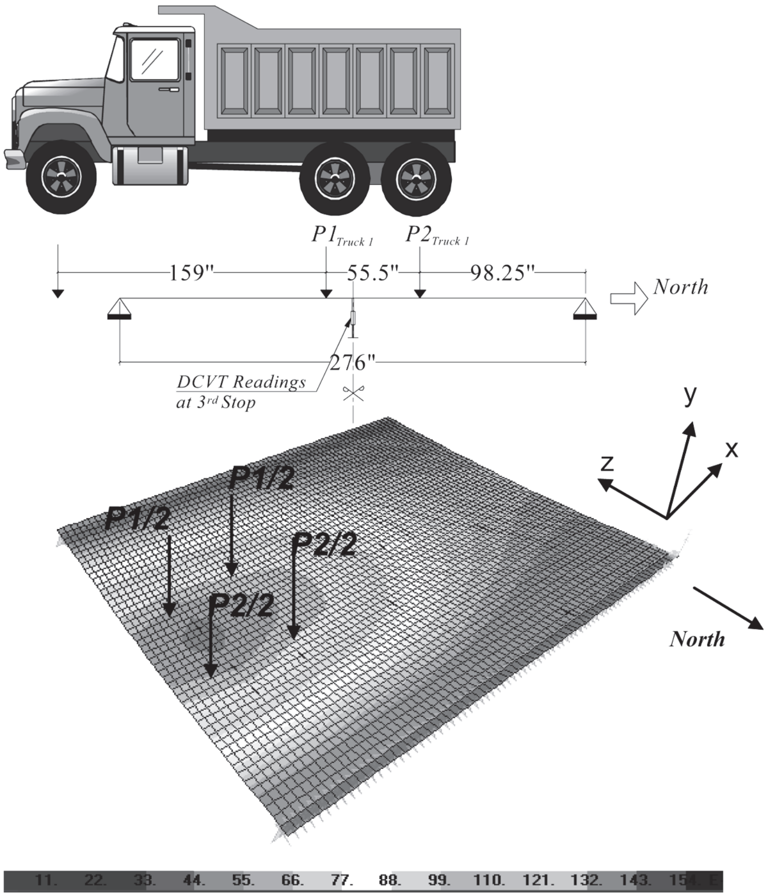


Fig. 10 Model 2: Deformed Shape of Panels with Shear Connectors (US Units; *1in x 10*$^{-3}$ *= 25.4 mm x 10*$^{-3}$)

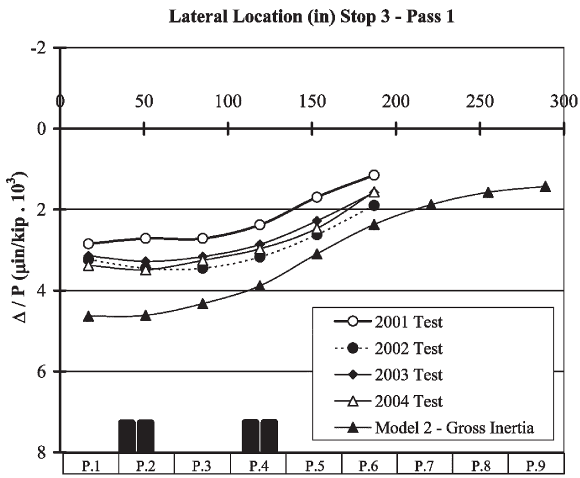


Fig. 11 FRP-RC Deck: Model 2 compared with the 4-Year Test
(US Units; *1in = 25.4mm, 1μin/kip= 5.7 μmm/kN*)