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MaineDOT

Transportation Research Division

Technical Report 14-09

Advanced Bridge Safety Initiative

Recommended Practices for Live Load Testing of Existing Flat-Slab Concrete Bridges– Task 5

Final Report – Task 5, December 2012

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16. Abstract (Limit 200 words)

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This document is a guide that provides additional recommendations on accepted AASHTO procedures and for live load testing of flat-slab concrete bridges. Sections 2 – 6 give specific recommendations in addition to those in the AASHTOs Manual for Bridge Evaluation (AASHTO 2008). Section 7 provides an example of a report for a live load test that was conducted on Bradford Bridge #3430 (Poulin 2012).

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ADVANCED STRUCTURES AND COMPOSITES CENTER

Recommended Practices for Live Load Testing of Existing Flat-Slab Concrete Bridges

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1 Background

Current AASHTO provisions for load rating flat-slab concrete bridges use the equivalent strip width method, which is regarded as overly conservative compared to more advanced analysis methods and field live load testing. It has been shown that live load testing of bridges can provide more accurate responses and can be used to calculate less conservative load rating factors. Amer et al (1999), Jáuregui et al. (2010), Poulin (2012) and Saraf (1998) have all demonstrated the benefits of non-destructive testing for the load rating of flat-slab concrete bridges.

This document is a guide that provides additional recommendations on accepted AASHTO procedures and for live load testing of flat-slab concrete bridges. Sections 2 – 6 give specific recommendations in addition to those in the AASHTOs *Manual for Bridge Evaluation* (AASHTO 2008). Section 7 provides an example of a report for a live load test that was conducted on Bradford Bridge #3430 (Poulin 2012).

2 Inspection and Theoretical Load Rating

In addition to recommendations in Section A8.2 of the *Manual for Bridge Evaluation,* a finite-element-based load rating should be performed prior to the live load test. This can be used to assess the need for a field live load test. If the bridge has a rating factor less than one, then minimum rating factors should be calculated for the entire bridge based on truck dimensions that closely represent the truck that will be used during the live load test. The location of the minimum rating factor and the location of the truck producing the minimum rating factor should be noted. Along with determining the minimum rating factor, a variety of different truck locations should be modeled to determine other locations that will produce the largest moments at specified locations along the bridge. This will ensure that the gauges placed on the bridge will be in locations that experience the largest strain.

3 Planning and Preparation

While planning and preparing for a live load test of a flat-slab concrete bridge, an instrumentation and test plan should be developed. This will include locating where gauges will be placed and determining truck positions to be used for the live load test. The locations of the gauges and the truck positions should be based on an analysis performed as per Section 2 of this report. Trucks should be placed to maximize the moment along the bridge and gauges should be placed at the locations of maximum moment for each truck pass. If possible, gauges should be placed not only on the bottom of the bridge but also at other locations through the cross section to allow the actual location of the neutral axis to be estimated from the load test strain data.

While preparing the plans for the live load tests a reference point on the bridge should be determined prior to the testing of the bridge to allow the relative location of truck and gauge positions to be easily measured. This reference point should be at an accessible location like a transverse edge of the bridge directly above the inside face of the support. In addition to what is stated here, Section A8.4 of the *Manual of Bridge Evaluation* should also be reviewed.

4 Execution of Load Test

In addition to the recommendations of Section A8.5 of the *Manual of Bridge Evaluation*, individual axle weights of the test vehicle should be measured along with the total truck weight. Axle spacing, transverse wheel spacing, and dimensions for each axle must be measured. If truck passes are run over the entire length of the bridge, wheel circumference needs to be measured to help determine the location of the truck along to bridge during the live load test. Measuring the wheel circumference should be done by marking a start position of a truck, marking the point of the wheel that is on the ground at the start, driving the truck for a specified number of full tire rotations, and then measuring the distance the truck has traveled. Given the wheel circumference, every time the truck travels the equivalent of one full tire revolution can be noted in the strain gauge data, automatically providing the truck position for every strain value. Intermediate positions of the truck can be interpolated assuming that the truck is driven at a constant speed. Along with the truck measurements, the strain gauges used in the live load test should be equipped with extensions whenever possible to ensure accurate average strain data for cracked sections.

5 Evaluation of Load Test Results

Along with the recommendations made in the *Manual for Bridge Evaluation* section A8.6, it is recommended that a finite-element model of the live load test be created. The finite-element models should use the true truck dimensions and weights placed at locations corresponding to the live load tests. Truck locations that produce the maximum effect in the bridge should be modeled and the resulting moments should be compared to the moments computed using the measured strains and bridge section properties.

To convert the strains from the live load test to moments there are two options. First, if gauges were placed on the top and bottom of the slab, the neutral axis location can be determined from the strains in the gauges on the top and bottom. The section properties can then be based on that neutral axis location, and the internal moment can be calculated from the strain. If gauges were not placed on the top of the slab, then section properties should be based on both a cracked section and an uncracked section. Moments can then be calculated based on both the section properties. To ensure the results are reasonable the predicted moment from the model should be within the limits of the moments based on cracked and uncracked section properties. The load rating for the structure should be computed per Section 8.8 of the *Manual for Bridge Evaluation*.

6 Reporting

Additional reporting beyond what is outlined in section A8.8 of the *Manual of Bridge Evaluation* should include bridge characteristics and properties and test truck weights, dimensions and locations. Plots showing either moment or strain vs. longitudinal truck positions should be provided. If many different live load tests are run, the test that provided the maximum effect, either moment or strain, should be presented in the report for each gauge used in the tests. Also if a model was created a description of the model along with comparison of the model to the live load test results should be provided in the report.

7 Example Load Test

This section documents the live load test of Bradford Bridge #3430 performed in October, 2011. There are some deviations from recommendations in these guidelines. These differences include not placing gauges on the top of the bridge to measure the location of the neutral axis, and failing to measure individual truck axle weights. These were not oversights, but either could not be achieved or resulted from poor communication. Since individual axle weights were not measured, average wheel distributions from similar trucks used in previous live load tests were used to apportion the vehicle weight to each axle.

7.1 Bridge Characteristics

The characteristics of Bradford Bridge #3430 are shown in Table 1. The concrete slab was assumed to have a compressive strength $f_c^{'}$ of 17.24 MPa, an elastic modulus of 19640 MPa, a Poisson's ratio of 0.19 and a unit weight of 2400 kg/m^3. The plans for the bridge were provided by the MaineDoT. The span length was taken as the distance between the centerline of each support.

The plans provided by the MaineDoT did not include the clear distance to the reinforcing or the area of steel in the transverse direction. A value of 51 mm was assumed for the clear distance to the reinforcing. Review of the plans of similar structures from the same era indicated clear distances of 25 mm to 51 mm. The area of steel in the transverse direction was assumed using a similar procedure. Transverse reinforcing details for similar bridges that were built around the same time were examined and it was found that two bridges had similar year of construction, lack of skew and span length. These structures had #5 bars transversely spaced at 0.24 m, which was assumed to be the transverse reinforcement for the Bradford Bridge #3430.

The longitudinal moment resistance and transverse moment resistance are also summarized in Table 1. The moment resistances were computed according to AASHTO *LRFD Bridge Design Specifications* (AASHTO 2010), and do not include the strength reduction factor ϕ . The cracking moment was computed assuming a composite section consisting of uncracked concrete and steel and a modular ratio n = 10.18. The tensile rupture strength of the concrete taken to be 2310 kPa based on AASHTO (2010) and the assumed value for $f_c^{'}$ of 17.24 MPa.

Table 1 – Characteristics of Bradford Bridge #3430

Length (m)	Width (m)	Slab Thickness (m)	Area of Steel – Longit. (cm2/m)	Area of Steel – Transv. (cm2/m)	Long. Moment Resistance M_n (kN-m/m)	Transv. Moment Resistance M_n (kN-m/m)
7.16	7.62	0.419	31.92	8.20	240.3	70.8

Bradford Bridge #3430 was load rated using both the conventional strip width method and FE analysis (Poulin 2012). The minimum operating load rating for the bridge is 0.482 per the conventional strip width method and 0.585 per the FE analysis, both of which were controlled by the HL-93 tandem. The HL-93 tandem loading also includes the design lane load as specified in AASHTO *Bridge Design Specifications* (2010).

7.2 Instrumentation Plan

Twenty two re-usable Bridge Diagnostics, Inc. (BDI) ST350 Intelligent Strain Transducers were placed on the bridge for the live load test. The locations of the gauges are shown in Figure 1 and Figure 2 for the gauges on the top and bottom of the bridge respectively. The wide range of gauge locations was chosen to provide a complete picture of the response of the structure and provide redundant measurements. Table 2 summarizes gauge locations and which gauges used extensions during the tests. The x and y locations are measured from the centerline of the support on the downstream right side of the bridge (bottom left of Figure 1 and Figure 2). Gauges were not placed on the top side of the interior part of the bridge (everywhere other than the curbs and rails) because this would have necessitated removal of the wearing surface in order to place the gauge directly to the top of the slab. The gauges that were placed on the top of the bridge are located on either the top of the inner curb or the top of the bottom railing as shown in Figure 3.

The top gauges that have *y*-locations of 0.23 m and 7.39 m were placed on the top of the bottom rail like gauge B3058 in Figure 3. The gauges with *y*-locations of 0.38 m and 7.24 m were located on the top of the inner side of the curb like gauge B3071 in Figure 3. Figure 4 shows a gauge with an extension (gauge length of 0.61 m, gauge B3070) along with a gauge that does not use an extension (gauge length of 0.08m, gauge B3057). Even though extensions are recommended when testing reinforced concrete they were not used for every gauge as the system only included six extensions. However, the most critical gauges located longitudinally at the bottom of the slab at mid-span (10, 12, 14, 16 and 18) all had extensions.

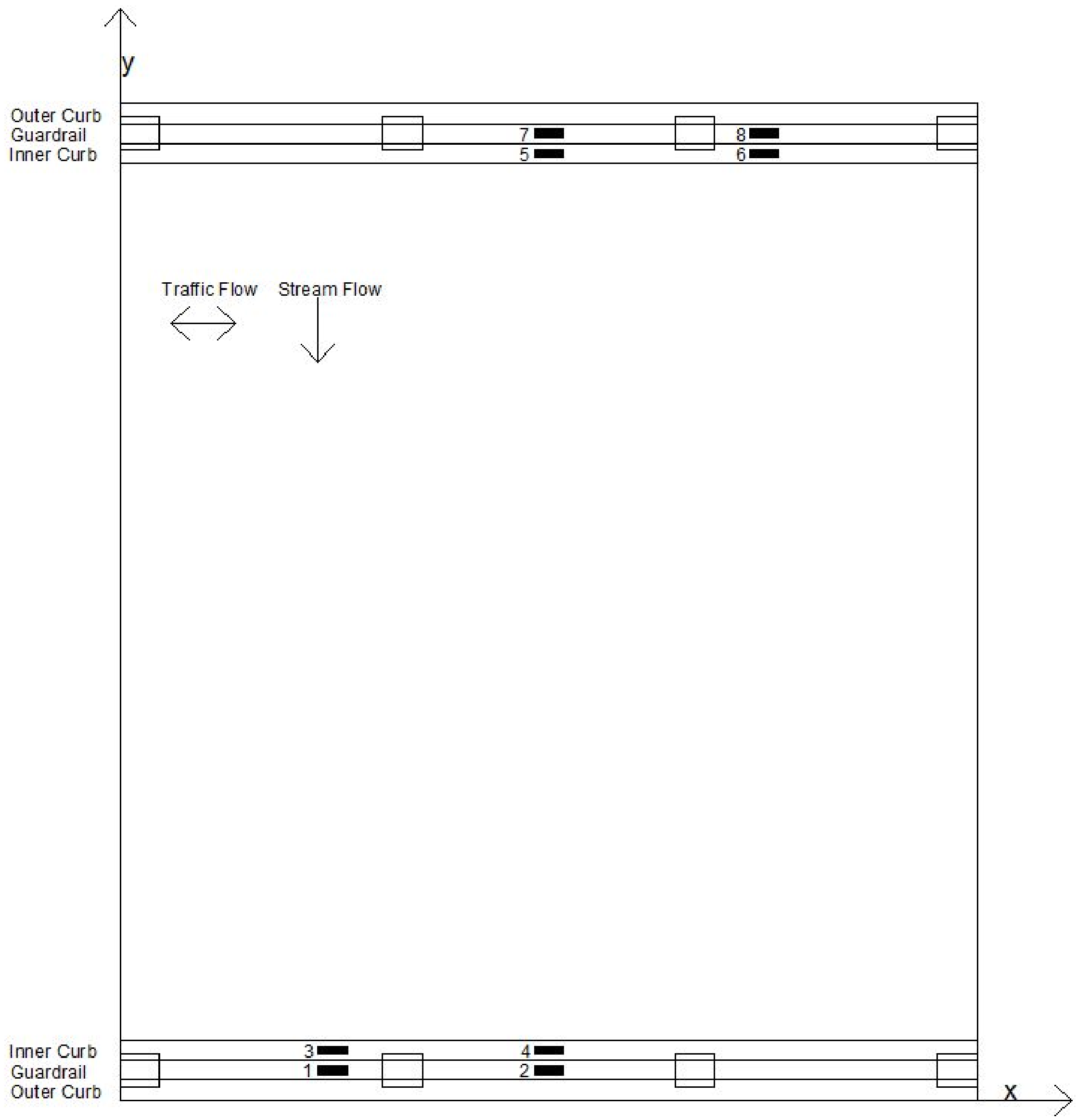

Figure 1 – Schematic of gauges located on the top of the bridge

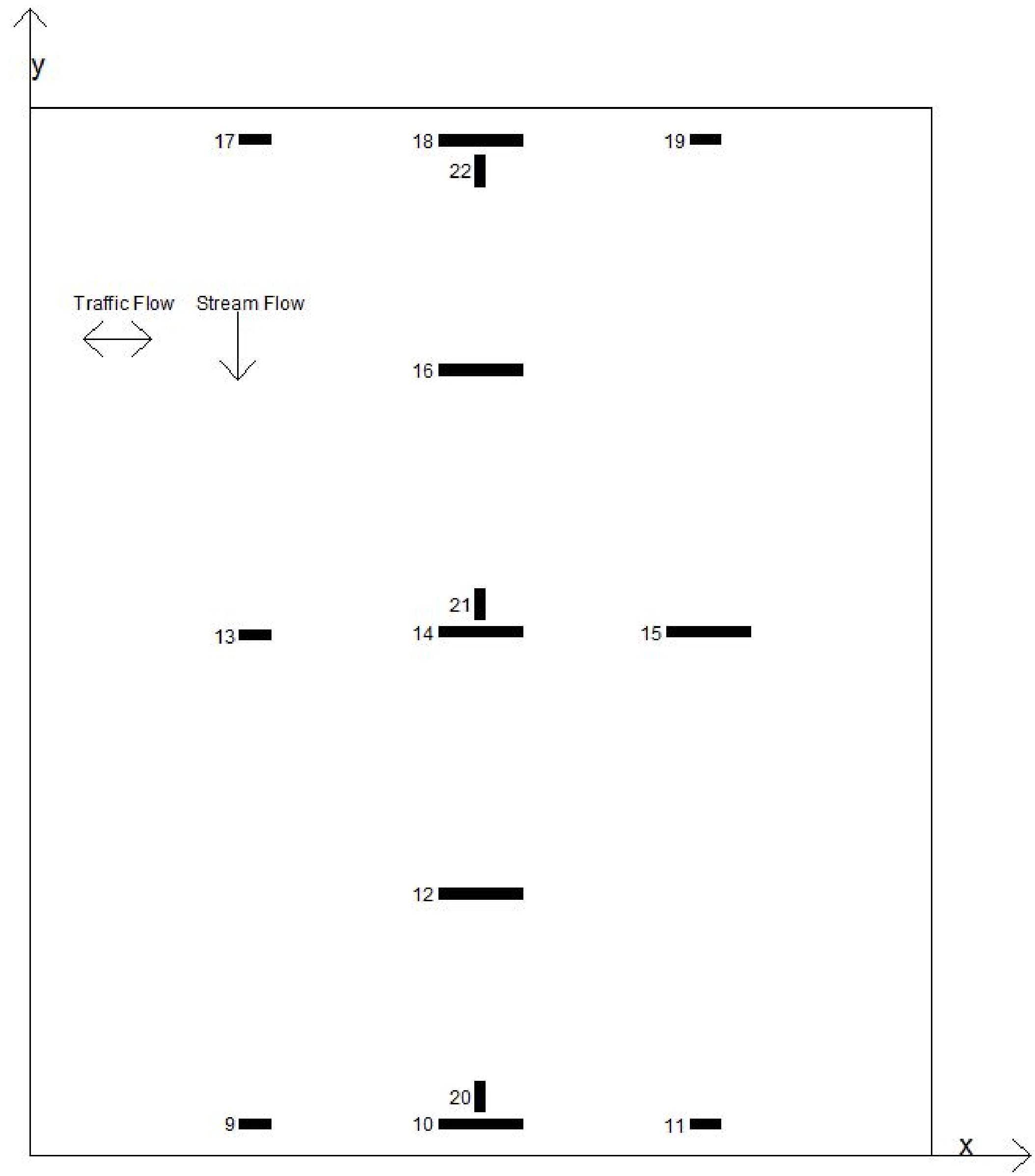

Figure 2 – Schematic of gauges located on the bottom of the bridge (longer gauges indicates an extension)

Table 2 – Location of each of the 22 gauges used in the live load test of Bradford Bridge #3430

Figure Reference Number	BDI Gauge Number	x – Location (m)	y – Location (m)	Top or Bottom	Gauge Direction	Extension?
1	B3058	1.96	0.23	Top	Longitudinal	No
2	B3065	3.58	0.23	Top	Longitudinal	No
3	B3071	1.96	0.38	Top	Longitudinal	No
4	B3055	3.58	0.38	Top	Longitudinal	No
5	B3056	3.58	7.24	Top	Longitudinal	No
6	B3069	5.23	7.24	Top	Longitudinal	No
7	B3066	3.58	7.39	Top	Longitudinal	No
8	B3061	5.23	7.39	Top	Longitudinal	No
9	B3072	1.96	0.23	Bottom	Longitudinal	No
10	B3070	3.58	0.23	Bottom	Longitudinal	Yes
11	B3068	5.23	0.23	Bottom	Longitudinal	No
12	B3076	3.58	1.91	Bottom	Longitudinal	Yes
13	B7073	1.96	3.81	Bottom	Longitudinal	No
14	B3062	3.58	3.81	Bottom	Longitudinal	Yes
15	B3060	5.23	3.81	Bottom	Longitudinal	Yes
16	B3064	3.58	5.71	Bottom	Longitudinal	Yes
17	B3063	1.96	7.39	Bottom	Longitudinal	No
18	B3059	3.58	7.39	Bottom	Longitudinal	Yes
19	B3075	5.23	7.39	Bottom	Longitudinal	No
20	B3057	3.58	0.32	Bottom	Transverse	No
21	B3074	3.58	3.9	Bottom	Transverse	No
22	B3067	3.58	7.28	Bottom	Transverse	No

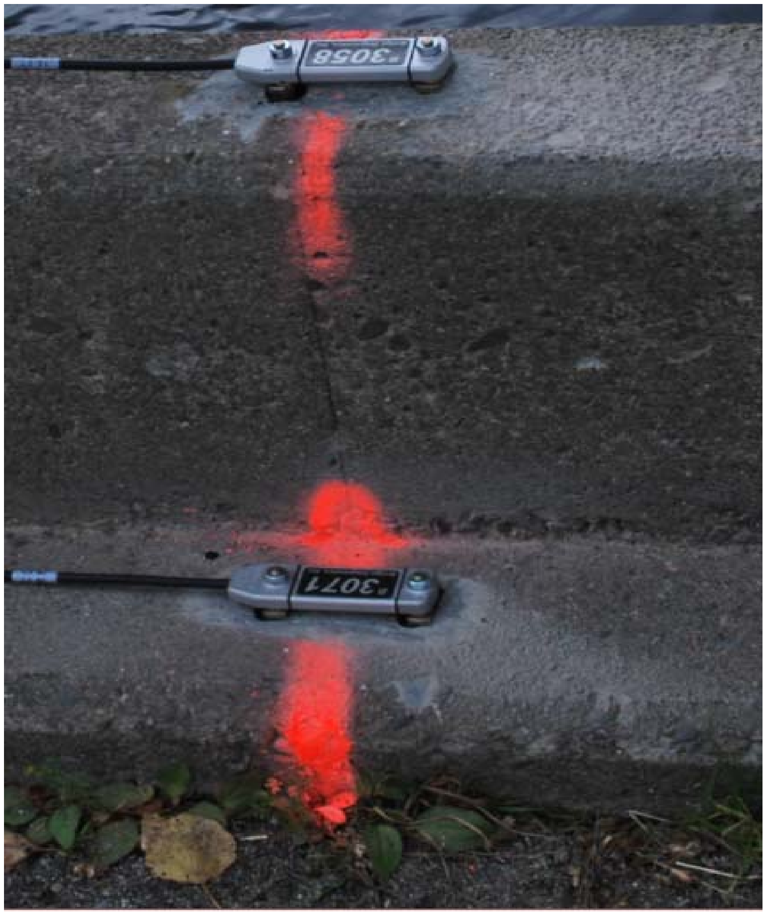

Figure 3 – Top gauge placements on the top of curb and top of the bottom rail

Figure 4 – Gauges placed with extension (Gauge B3070) and without extension (Gauge B3057)

7.3 Truck Information

The Bradford Bridge live load test was conducted with two trucks provided by the MaineDoT. The total weight of each individual truck was measured. The length of one tire revolution was determined for each truck, so the longitudinal position of the truck could be determined throughout the test. This was done by measuring the distance that truck moves for nine full tire revolutions. Both of the trucks traveled 29.96 m. Therefore one tire revolution covered a distance of 3.33 m. Figure 5 and Figure 6 show the distances between each of the axles and individual wheels for MaineDoT trucks #1 and #2 respectively. The measured width of each wheel was 228.6 mm.

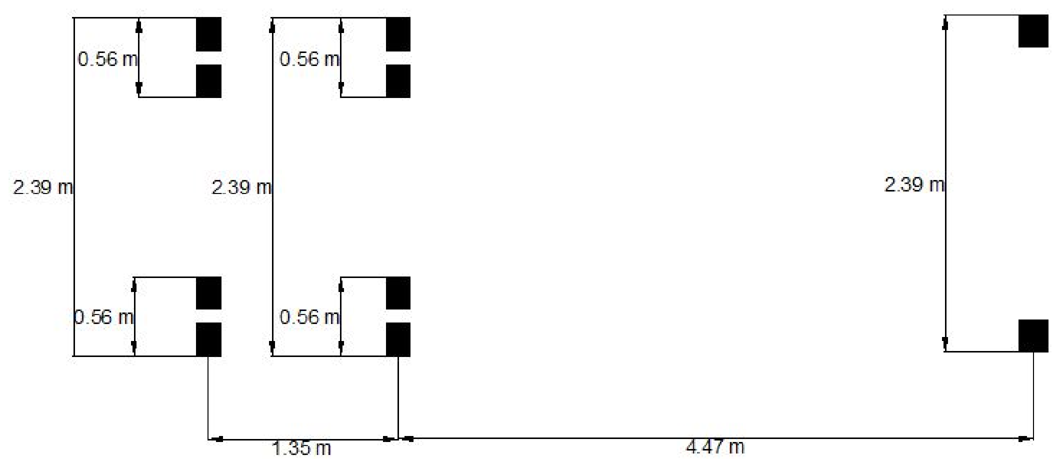

Figure 5 – Wheel spacing for MaineDoT Truck #1

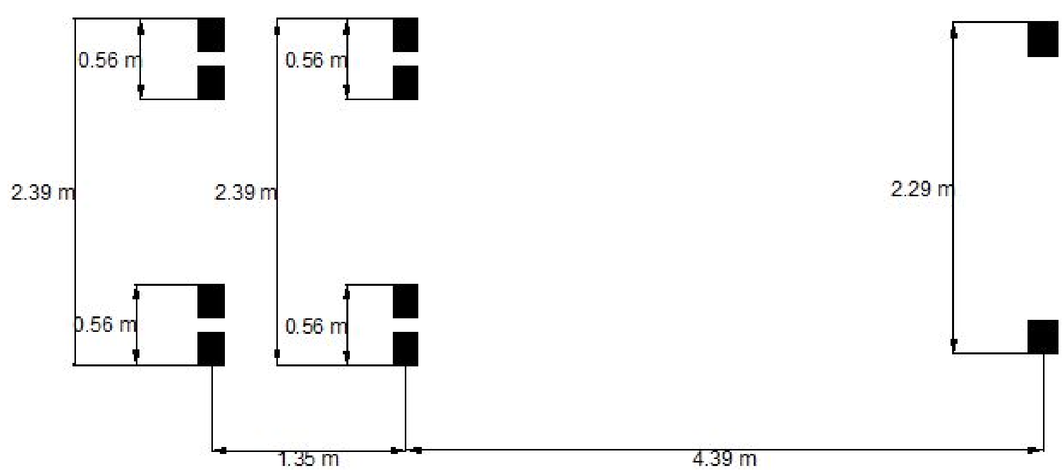

Figure 6 – Wheel spacing for MaineDoT Truck #2

The weights of each individual axle were not recorded, but were estimated by averaging axle weights of five separate MaineDoT trucks having the same axle configuration and nominal gross vehicle weight which were used during previous live load performed for the MaineDOT by the University of Maine. The truck axle weights and total truck weights for each of the trucks that have been used previously are shown in Tables 3 – 6. The values in Table 3 are the average of two trucks , while Tables 4 – 6 each contain data for only one truck..

Table 3 – Axle positions and weights for the average of two MaineDoT Trucks used in first Neal Bridge live load test

Axle Position (mm from front axle)	Weight of Each Axle (kN)	Percentage of Total Weight (kN)
0	75.84	26%
4521.2	112.32	37%
5918.2	108.31	37%
Total Weight	296.47	

Table 4 – Axle positions and weights for the MaineDoT Truck used in second Neal Bridge live load test

Axle Position (mm from front axle)	Weight of Each Axle (kN)	Percentage of Total Weight (kN)
0	63.61	26%
4826.0	88.96	37%
6197.6	88.96	37%
Total Weight	241.54	

Table 5 – Axle positions and weights for the MaineDoT Truck used in Fairfield Bridge live load test

Axle Position (mm from front axle)	Weight of Each Axle (kN)	Percentage of Total Weight (kN)
0	65.90	21%
4521.2	122.20	40%
5994.4	120.30	39%
Total Weight	308.40	

Table 6 – Axle positions and weights for the MaineDoT Truck used in Coplin Plantation live load test

Axle Position (mm from front axle)	Weight of Each Axle (kN)	Percentage of Total Weight (kN)
0	57.50	20%
4394.2	114.10	40%
5753.1	113.90	40%
Total Weight	285.50	

As seen in Tables 3 – 6, all the trucks have very similar axle weight distributions. The front axle is between 20-26% of the total truck weight while the back axles are all between 37-40%. Since all these trucks have the same relative distribution, the average of the axle distributions were used to determine the axle weights for the trucks used in the Bradford live load test as given in Table 7 and Table 8.

Table 7 – Axle positions and weights for the MaineDoT Truck #1 used in Bradford Bridge live load test

Axle Position (mm from front axle)	Axle Weight (kN)	Percentage of Total Weight
0	58.03	23%
4470.4	95.88	38%
5816.6	98.40	39%
Total Weight	252.30	

Table 8 - Axle positions and weights for the MaineDoT Truck #2 used in Bradford Bridge live load test

Axle Position (mm from front axle)	Axle Weight (kN)	Percentage of Total Weight
0	59.54	23%
4394.2	98.38	38%
5740.4	100.97	39%
Total Weight	258.89	

7.4 Test Truck Positions

Seven different truck passes were run during the live load test of the Bradford Bridge. Four different transverse truck positions were used during the test. Tests 1 and 2 used the same transverse truck position, tests 3 and 4 used the same transverse truck position and tests 5 and 6 used the same transverse truck positions. These truck positions were run twice to ensure the results were consistent and repeatable.

The truck position for tests 1 and 2 are shown in Figure 7. The outside of the front tire was placed 0.61 m away from the inside face of the downstream side of the inner curb. Figure 8 shows the truck position for tests 3 and 4. This truck position was the opposite of test 1 and 2, with the outside of the front tire placed 0.61 m away from the inside face of the upstream side curb. Both of the tests used truck #2 provided by the MaineDoT because it was the larger of the two trucks. These positions placed the truck as close as possible to the edge of the bridge according to the *Manual for Bridge Evaluation* (AASHTO 2008). Also, since the striped lane width on the Bradford Bridge was only 2.74 m, and the truck was 2.29 m wide, only one truck pass was run for each lane. Tests 1-4 were run to maximize the strains in the gauges located on the outside edges of the bridge.

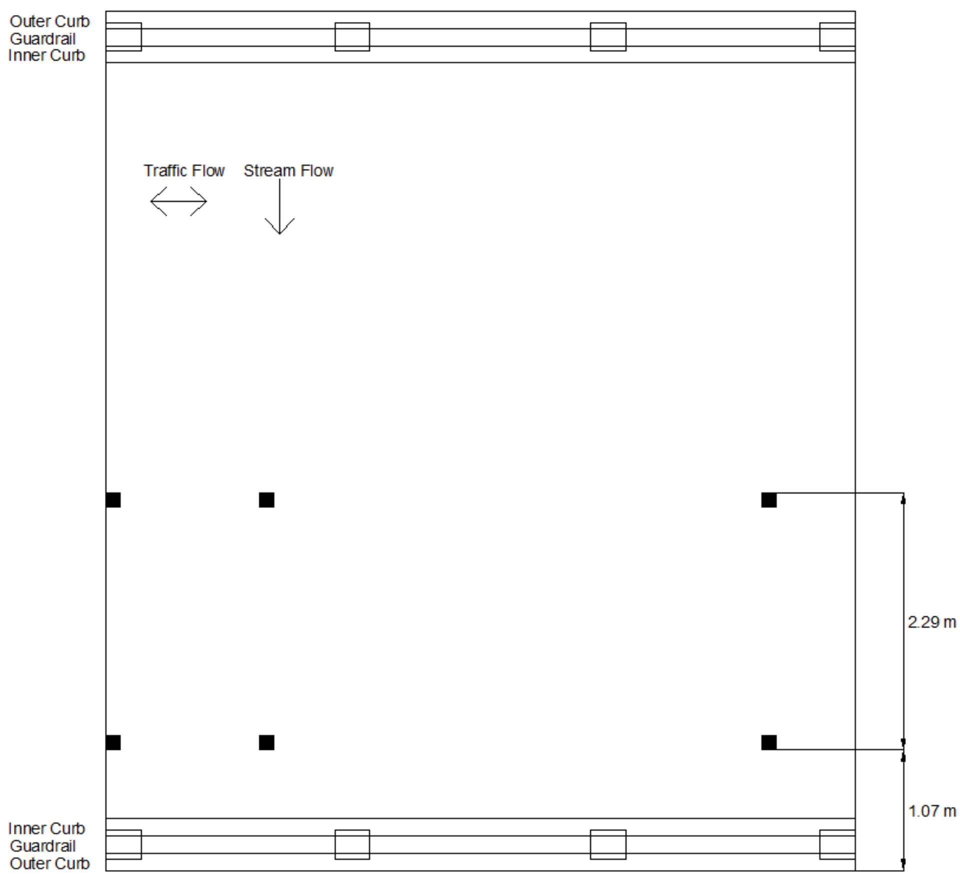

Figure 7 – Position of truck during live load tests 1 and 2

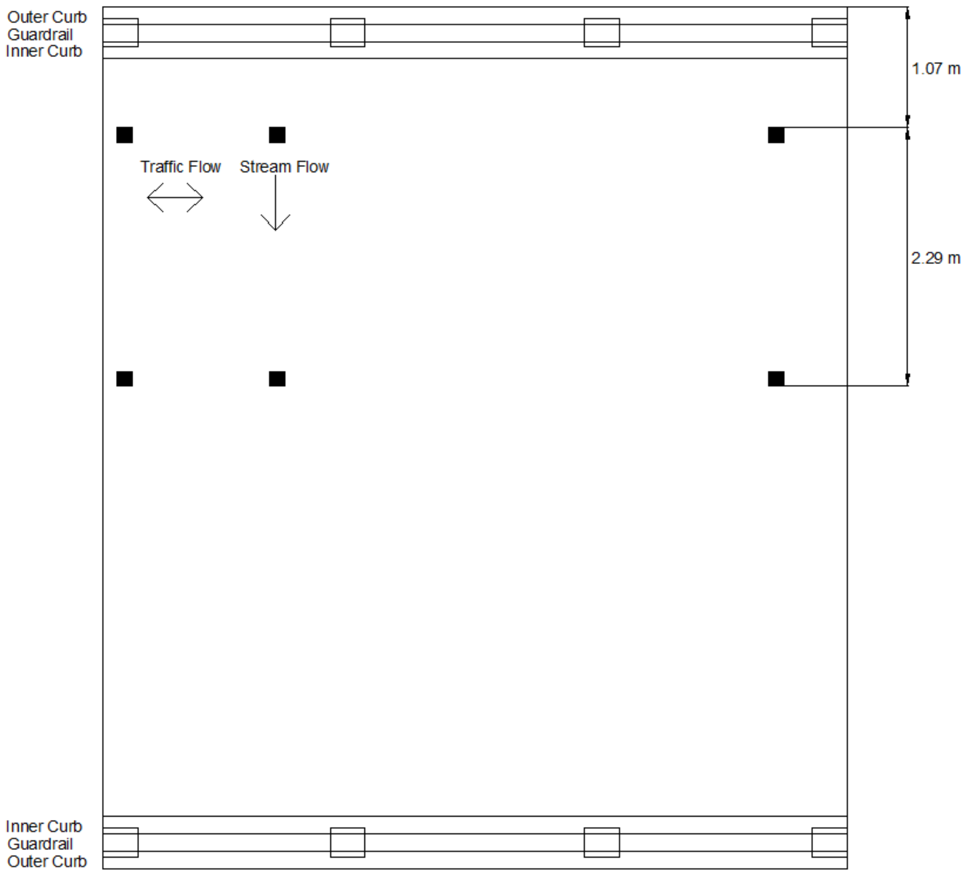

Figure 8 – Position of truck during live load tests 3 and 4

Figure 9 shows the transverse truck placement for tests 5 and 6. These tests were conducted with both the trucks on the bridge, truck # 2 placed on the upstream side and truck # 1 on the downstream side. Truck #2 was placed 0.61 m away from the upstream inner curb as in test 3 and 4. With truck #1 placed 1.22 m away from truck #2, the closest transverse truck spacing according to AASHTO's *Manual for Bridge Evaluation* (AASHTO 2008), the outside of truck #1's tires were 0.31 m from the inner face of the downstream curb. The reason the test was not done the other way (truck #2 placed 0.61 m from the downstream face of the inner curb and truck #1 placed 1.22 m away from the inner tires) was that it only shifted the trucks by 0.31 m, which was not expected to significantly affect strains.

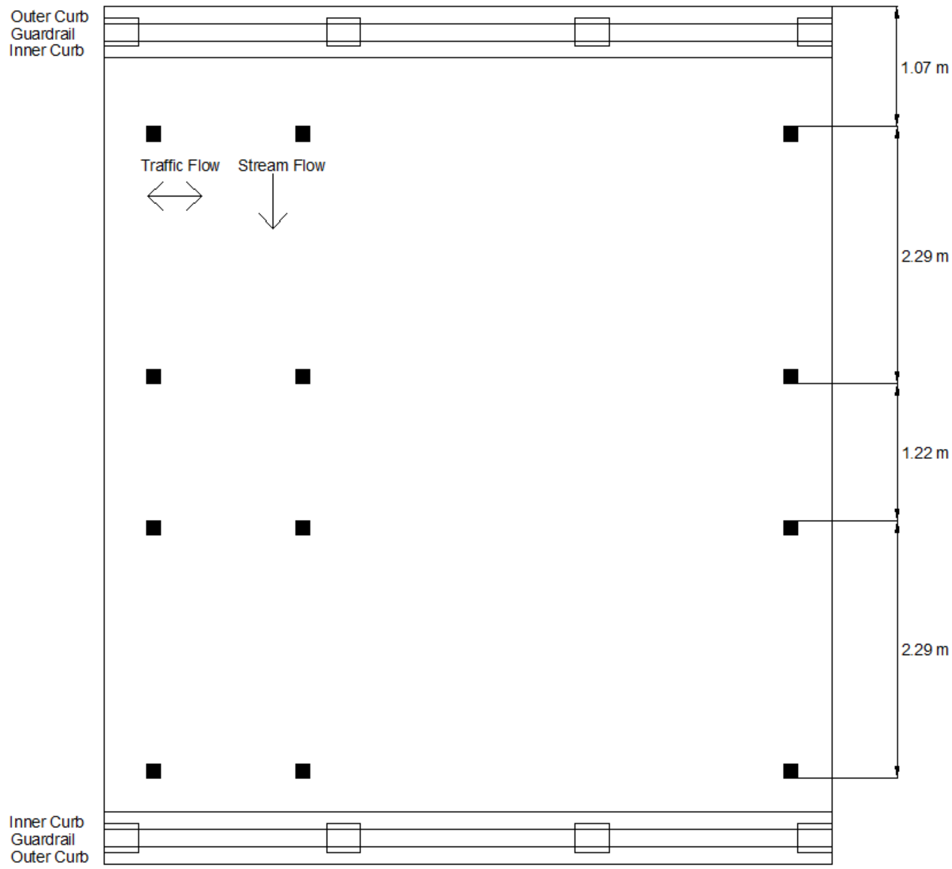

Figure 9 – Position of truck during live load tests 5 and 6

Figure 10 shows the placement of the truck during test #7. The centerline of the truck was positioned over the transverse centerline of the bridge. Truck #2 was used in this test because it was heavier then truck #1. This test was run to maximize the strains over the transverse centerline of the bridge when only one truck was placed on the bridge.

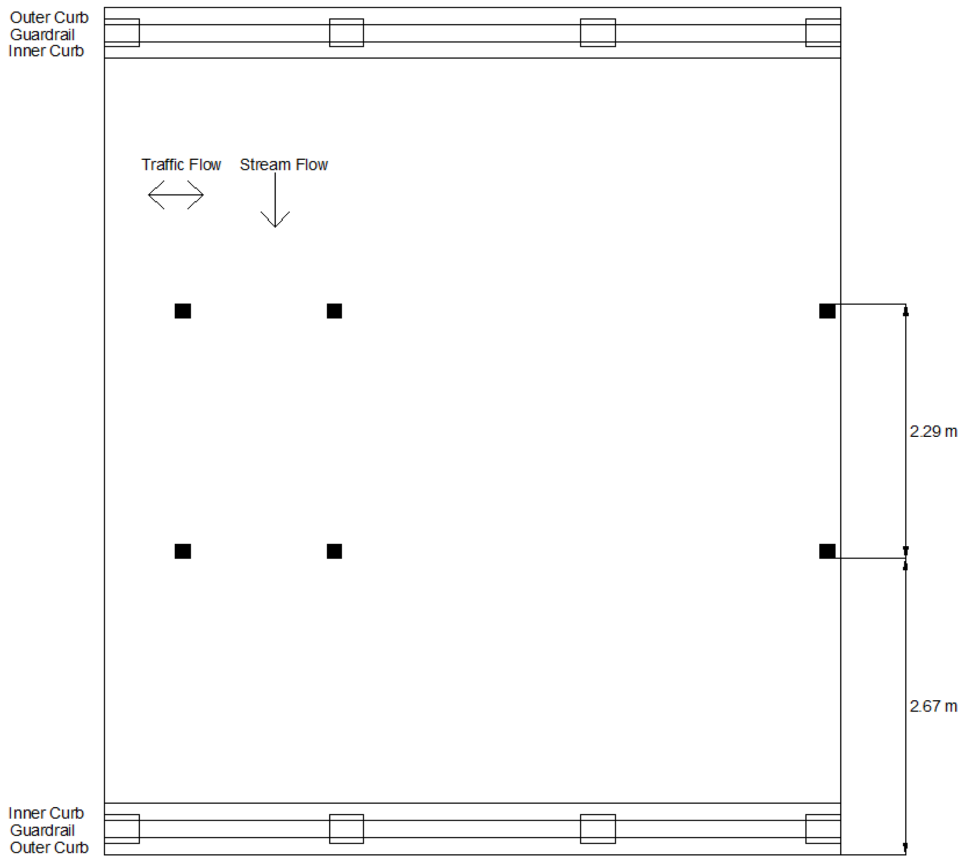

Figure 10 – Position of truck #2 during live load test 7

7.5 Resulting Strains for Each Live Load Test

From each of the seven tests the resulting strains in the twenty two gauges were recorded. Not only were the gauges with extensions (gauges 10, 12, 14, 15, 16 and 18) the critical gauges recording the largest strains, but they also provided the most consistent and reliable results. Therefore results for only those gauges will be examined further. Figure 11 - Figure 13 are plots of strains vs. truck position during the live load test that produces the maximum strain for each individual gauge with an extension for only one truck of loading (tests 1 – 4 and test 7). Figure 14 - Figure 16 are plots of strain vs. truck position for the tests that caused the maximum strain in each individual gauge with an extension when two trucks were used in the live load test (tests 5 and 6). The position of the front axle was measured from the centerline of the support where the truck started moving.

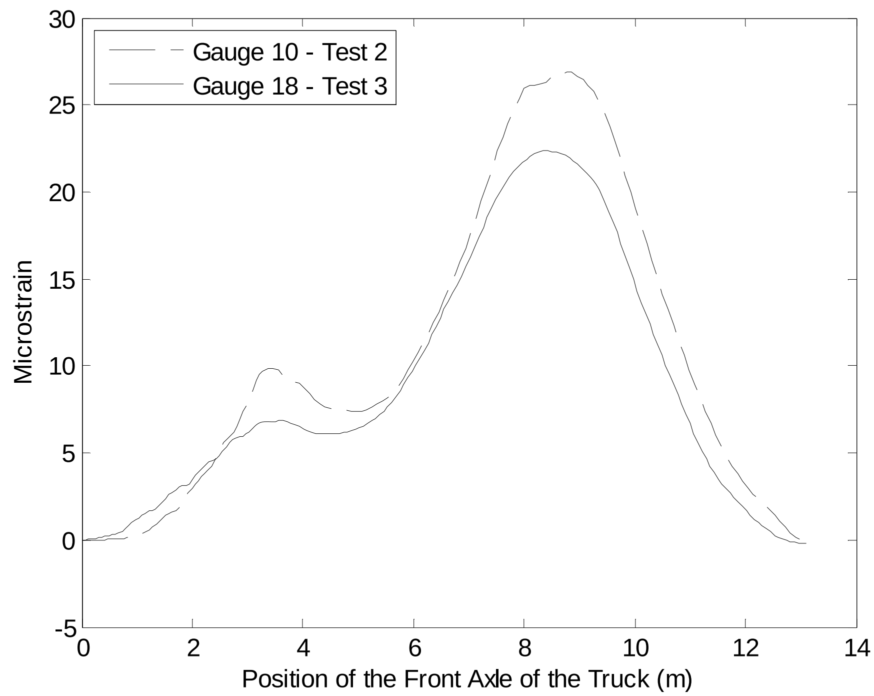

Figure 11 - Worst one-truck loading case for gauges located under the curb at centerline span

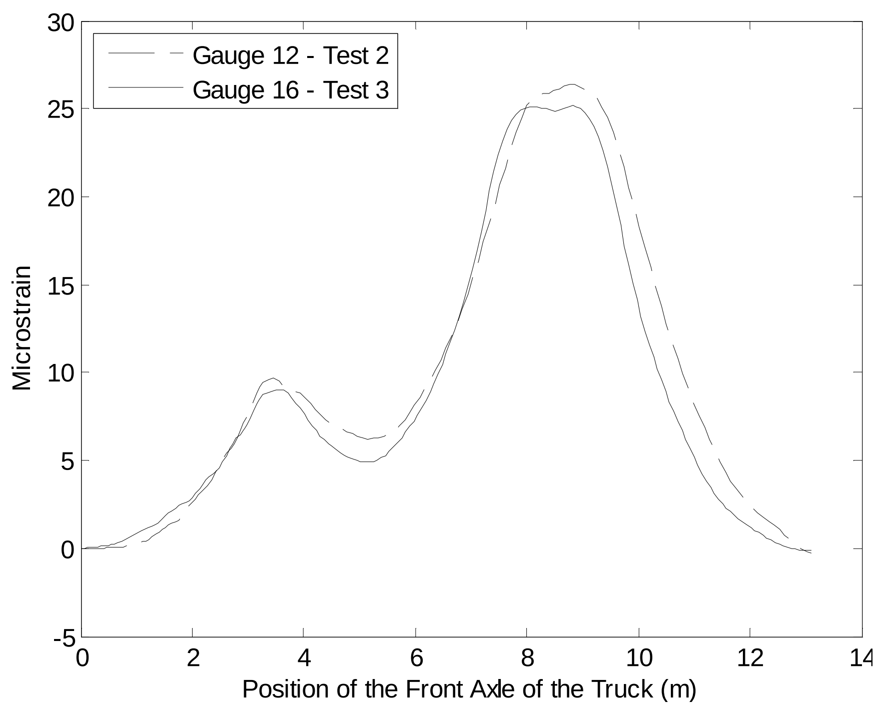

Figure 12 - Worst one-truck loading case for bottom gauges located at centerline span at the transverse quarter points

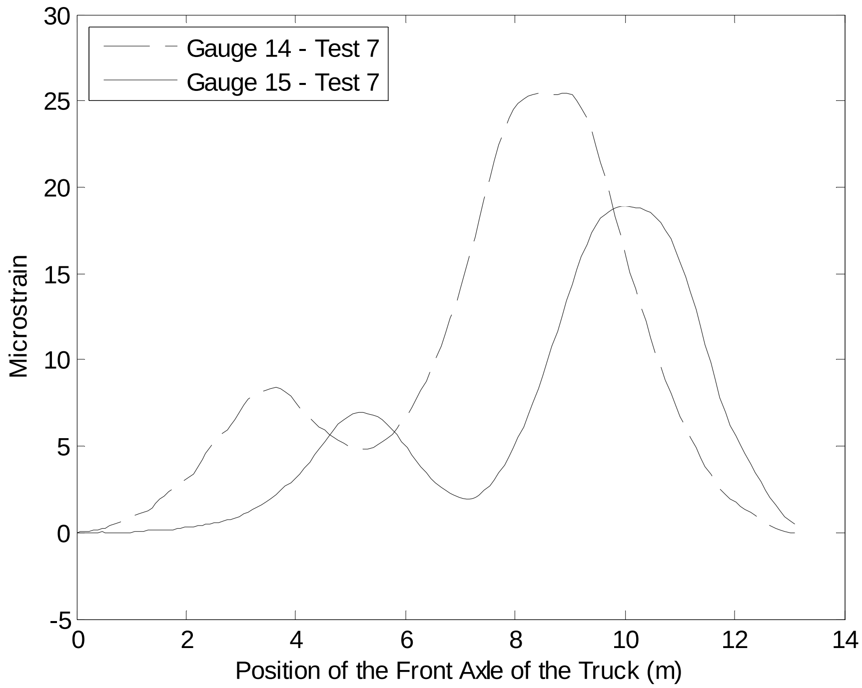

Figure 13 – Worst one-truck loading case for bottom gauges located along the transverse centerline of the bridge

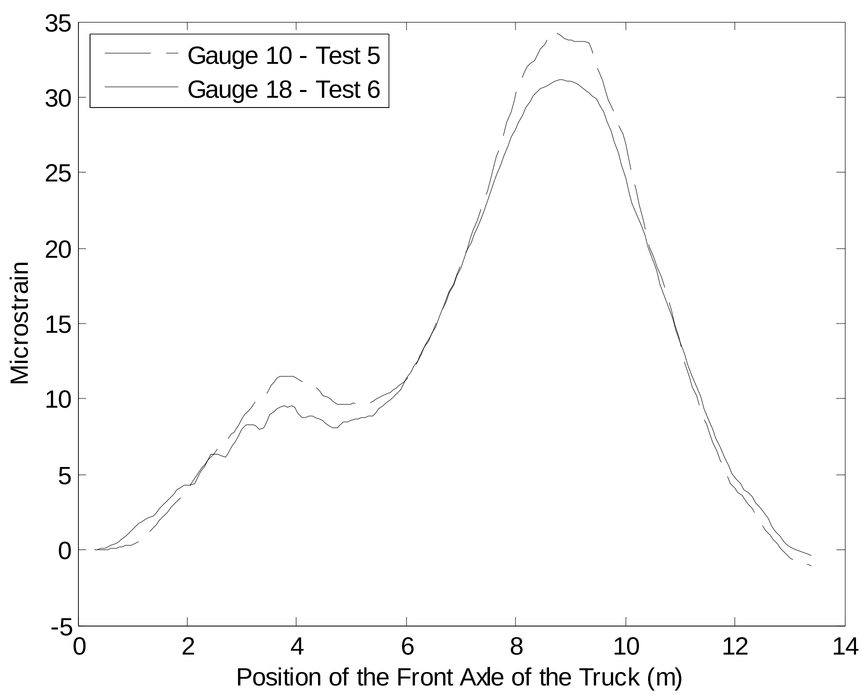

Figure 14 - Worst two-truck loading case for gauges located under the curb at centerline span

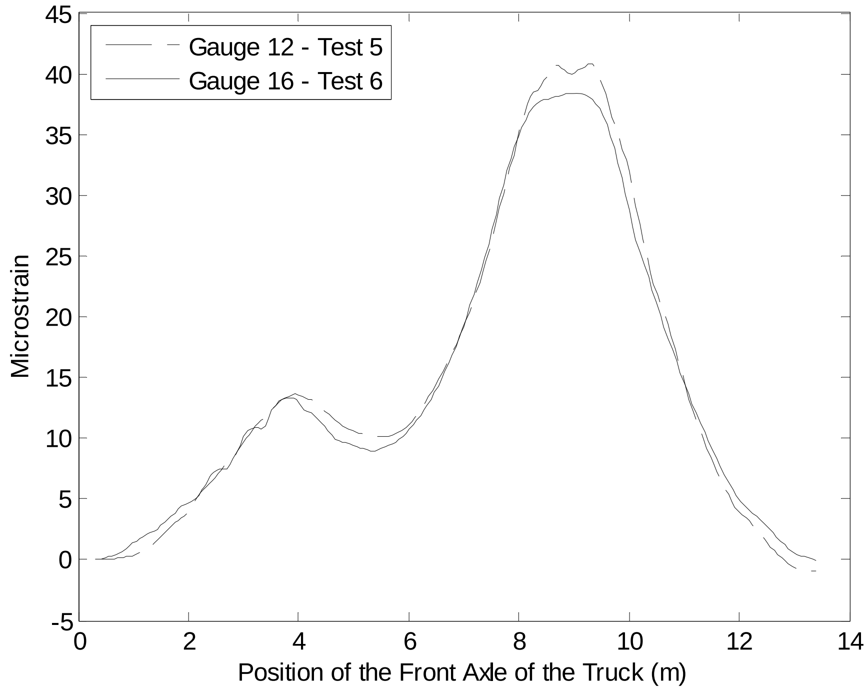

Figure 15 - Worst two-truck loading case for gauges located at centerline span at transverse quarter points

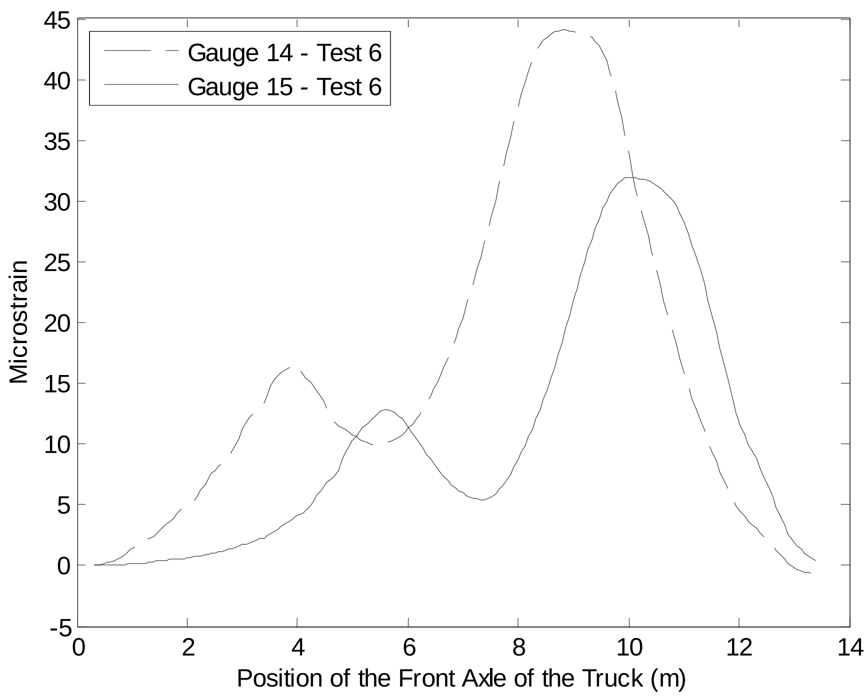

Figure 16 – Worst two-truck loading case for gauges located along the transverse centerline of the bridge

7.6 Discussion of Live Load Test Results

As can be seen in all the plots the results are consistent for similar gauges. All peaks of symmetrically located gauges are similar, sometimes with the only major difference being where the peaks occur. These differences in the location of the peaks are caused by the longitudinal position of the gauge, either the gauge being located at the quarter point close to the initial position of the truck or the opposite quarter point . The small initial peak in the data is caused by the front axle passing over the longitudinal position of the gauge while the larger peak is caused by the rear axles traveling over the longitudinal position of the gauge.

It must be noted that the truck positions corresponding to each strain value are not exact. The truck position was measured based on revolutions of the tire. A marker was placed on the tire and every time that marker went one full cycle it was recorded manually with the strain data. The truck was assumed to be traveling at a constant rate between recorded wheel revolutions.

7.7 Modeling Field Live Load Tests

After the live load tests were completed the results were compared to finite element predictions developed using the software SlabRate (Poulin 2012), which relies on 8-noded shear deformable plate elements. FE model predictions were compared with the live load test results to assess the accuracy of the FE modeling strategy. The FE mesh used for the model comparisons had 32 longitudinal and 24 transverse elements, and is shown in Figure 17. While the convergence studies presented by Poulin (2012) indicate that a coarser mesh gives sufficient accuracy, the larger number of elements was chosen so that the strain gauge locations coincided with element corners at critical gauges 12, 14 and 16. Because the 8-noded quadratic plate element can only capture a bi-linear variation in moment over its area, peak strains will always occur at element corner nodes. Additionally, the longitudinal position of one corner node was shifted forward to coincide with the location of gauge 15 as shown in Figure 17.

It is also important to note that the Bradford bridge has a thickened concrete curb and barrier. Such a curb and barrier can increase the stiffness of the structure and reduce bending moments in interior portions of the slab. Inspection of the bridge indicated that the curb was cast monolithically with the slab, and that the lower portion of the barrier was rigidly attached to the slab. The barrier rails were not cast monolithically with the barrier posts, and the posts and barrier rails were therefore not assumed to affect slab response significantly. To capture the effect of the thickened curb and lower portion of the barrier in the FE model, the two outer rows of elements in the mesh had the same 457 mm width as the curb, and were assigned a thickness of 785 mm to give them a bending moment of inertia equal to that of the section consisting of the slab, curb and lower portion of the barrier. The remainder of the slab was assigned a constant thickness across its entire width. Consistent with the manner in which live load moments were inferred from strains, the thickness of the interior slab elements was taken as 521 mm, the total thickness of the structural slab and concrete wearing surface. Each individual tire was placed on the bridge so the tire sizes and weights corresponded to those of the actual truck that was used in the live load test. The assumed tire inflation pressure of 620.5 kPa and the measured tire width were used to determine the contact length of each wheel.

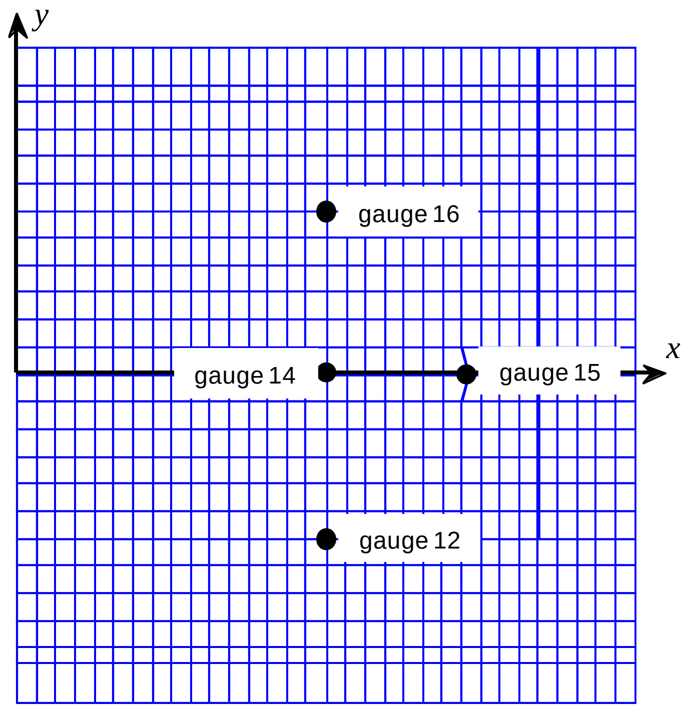

Figure 17 – SlabRate FE Mesh of Bradford Bridge

7.8 Live Load Test Calculated Moments

For ease of comparison with the FEA results, the measured live load strains were converted to longitudinal bending moments. This was accomplished using Equation 1, which follows the procedure given by Jáuregui et al. (2007).

$$M_{LL} = \varepsilon_{LL} E_c S \qquad \text{Equation 1}$$

In Eq. 1, M_{LL} is the live load moment, ε_{LL} is the measured live load strain, E_c is the concrete elastic modulus computed as 19,640 MPa per AASHTO (2010), and S is the bottom fiber section modulus of the composite reinforced concrete slab, where the steel is transformed into an effective concrete area using the modular ratio $n = E_s/E_c$. When interpreting the field data, the calculation of S is typically based on either a cracked or uncracked concrete section (Jáuregui et al. 2007), where an uncracked section modulus

S_{uncr} is used if the measured strains indicate that the concrete is unlikely to be cracked, and S_{cr} is appropriate for larger measured strains which indicate the section has experienced significant tensile cracking. For the Bradford bridge, S_{cr} was computed to be 135 cm^3/cm assuming that the concrete cannot carry tension (i.e. all concrete below the neutral axis was cracked). For details on calculating S_{cr}, the reader is referred to a text on concrete design (e.g. Wight and MacGregor 2009, pp. 414-416). The value of the uncracked section modulus S_{uncr} was computed to be 526 cm^3/cm, which is approximately 16% greater than the section modulus of the gross concrete section due to the presence of the reinforcing.

To assess whether the Bradford bridge behaved as a cracked or uncracked section, the largest live load moment inferred from the measured strains using Equation 1 with S_{uncr} was compared with the moment M_{cr} expected to initiate flexural cracking. Lower and upper bounds on M_{cr} were computed using Equation 2 below.

$$M_{cr} = f_r S_{uncr} \qquad \text{Equation 2}$$

The lower bound was computed to be 137 kN-m/m taking the concrete modulus of rupture f_r as 2.61 MPa per Section 5.7.3.6.2 of AASHTO (2010). An upper bound value of M_{cr} = 212 kN-m/m was computed using a value of f_r equal to 4.04 MPa determined per Section 5.7.3.3.2 of AASHTO (2010). For comparison with expected load effects, M_{cr} was taken as 175 kN-m/m, the average of the upper and lower bound values.

The maximum strain measured during the live load tests was 44.08 με (gauge 14, live load test 6). The live load moment M_{LL} corresponding to this strain computed from Eq. 1 based on an uncracked section is 45.5 kN-m/m, and the maximum expected dead load moment M_{DL} in the Bradford bridge is 88 kN-m/m based on a SlabRate FE analysis accounting for the weight of the slab, curbs and railings. The sum of the inferred live load moment and dead load moment is 133.5 kN-m/m, which is 24% less than the average estimate of M_{cr} .

To increase $M_{LL} + M_{DL}$ to the 175 kN-m/m average estimate of M_{cr} would require a live load moment of 175 – 88 = 87 kN-m/m. Based on the average tandem axle weight of 197 kN used in the live load tests, this implies that significant flexural cracking of the Bradford bridge would not occur until the bridge was simultaneously loaded with a pair of tandem axles weighing ${}^{87}\!/_{45.5} \times 197 = 377$ kN each located at or near mid-span. To put this value in perspective, a 377 kN tandem axle is approximately 71% heavier than the AASHTO HL-93 design tandem (AASHTO 2010). Based on these comparisons and the fact that the Bradford bridge is on a secondary road in a rural area that does not routinely see heavy truck traffic, it is unlikely that the Bradford bridge has experienced significant service load cracking. Live load moments were therefore inferred from strain using S_{uncr} in Eq. 1.

7.9 Comparison of the FE-predicted and Live Load Test Moments

Direct comparisons of the FE-predicted moments and M_{LL} values inferred at the interior gauge locations 12, 14, 15 and 16 for all truck positions during each live load test are given in Figure 18 through Figure 21. At all gauge locations, an initial peak bending moment was observed and predicted when the steer axle passed over the longitudinal location of the gauge, followed by a larger peak moment when the rear tandem axles passed over the gauge.

For load tests 5 and 6, where the largest values of ε_{LL} were recorded, the FE model over-predicted average peak moments recorded during these two tests by 44% and 47% at gauges 12 and 16, respectively, and over-predicted moment by 36% and 53% at gauges 14 and 15, respectively. The peak FE model predictions deviated from measured M_{LL} values inferred from the test data by -45% on average at all gauge locations for tests 5 and 6. This under-prediction of measured results indicates that the FE model is likely conservative. One reason for the model's under-prediction could be taking the slab span from centerline support to centerline support as opposed to using the clear span measured between the faces of the supports. Further, while the inspection of the structure did not indicate any restraint of the slab at the abutments, horizontal restraint of the slab ends could have resulted in some arching action not captured by the FE model which would tend to reduce field-measured slab tensile strains. Additionally, there could have been rotational restraint caused by friction between the abutments and slab ends. Overall, however, the FE model conservatively predicted measured response, and is a suitable tool for conservatively assessing the capacity of flat-slab concrete bridges.

The effect of the thickened slab edge designed to capture the stiffening provided by the integral curb and lower portion of the barrier was also examined with the FE model. To assess this effect, a parallel set of simulations was run with the same FE mesh having a constant slab thickness of 521 mm. This FE model with a constant slab thickness predicts an increase in live load moments for the two-truck loading case at gauge locations 12, 14, 15 and 16 of approximately 28%, 22%, 19% and 27%, respectively. These increases are relative to the moments predicted by the FE model with the thickened edge elements that account for the curb and barrier, and demonstrate the significant effect of the curb and barriers on slab bending response. The moments are lower for the model with the curb and barrier because these stiffer elements tend to draw moment away from the thinner interior region of the slab where the gauges are located.

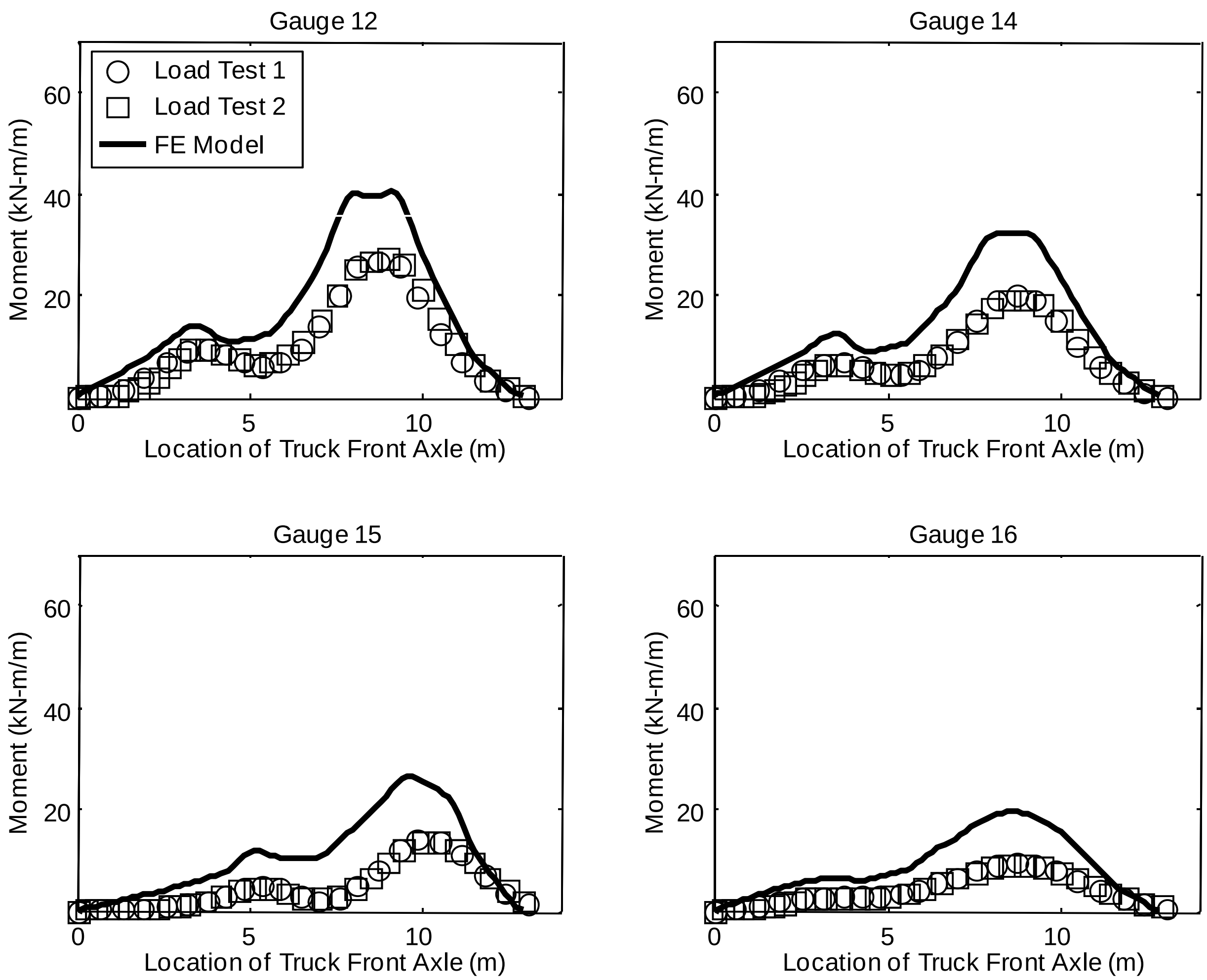

Figure 18 -- Test-derived and FE-predicted slab moments for live load tests 1 and 2

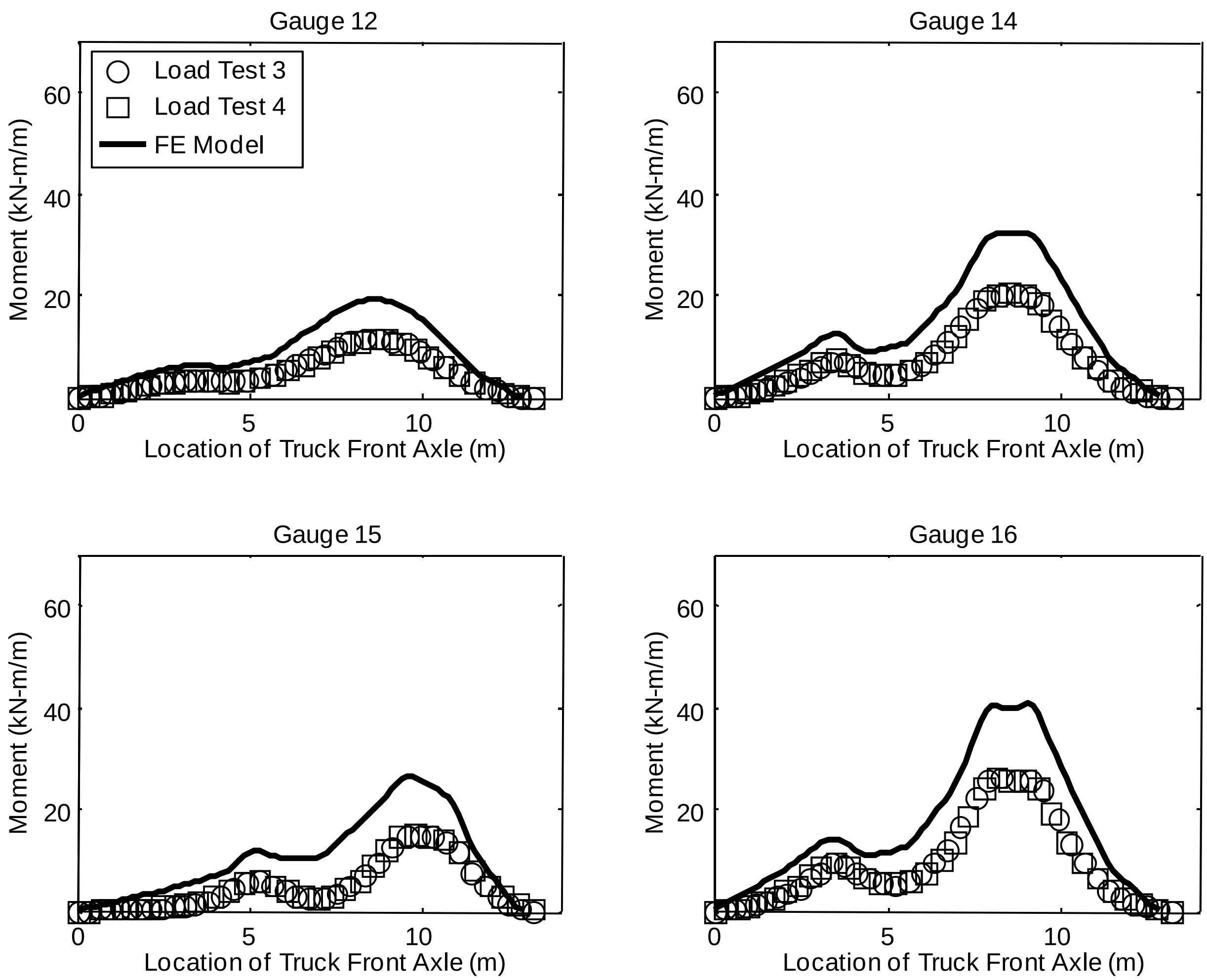

Figure 19 -- Test-derived and FE-predicted slab moments for live load tests 3 and 4

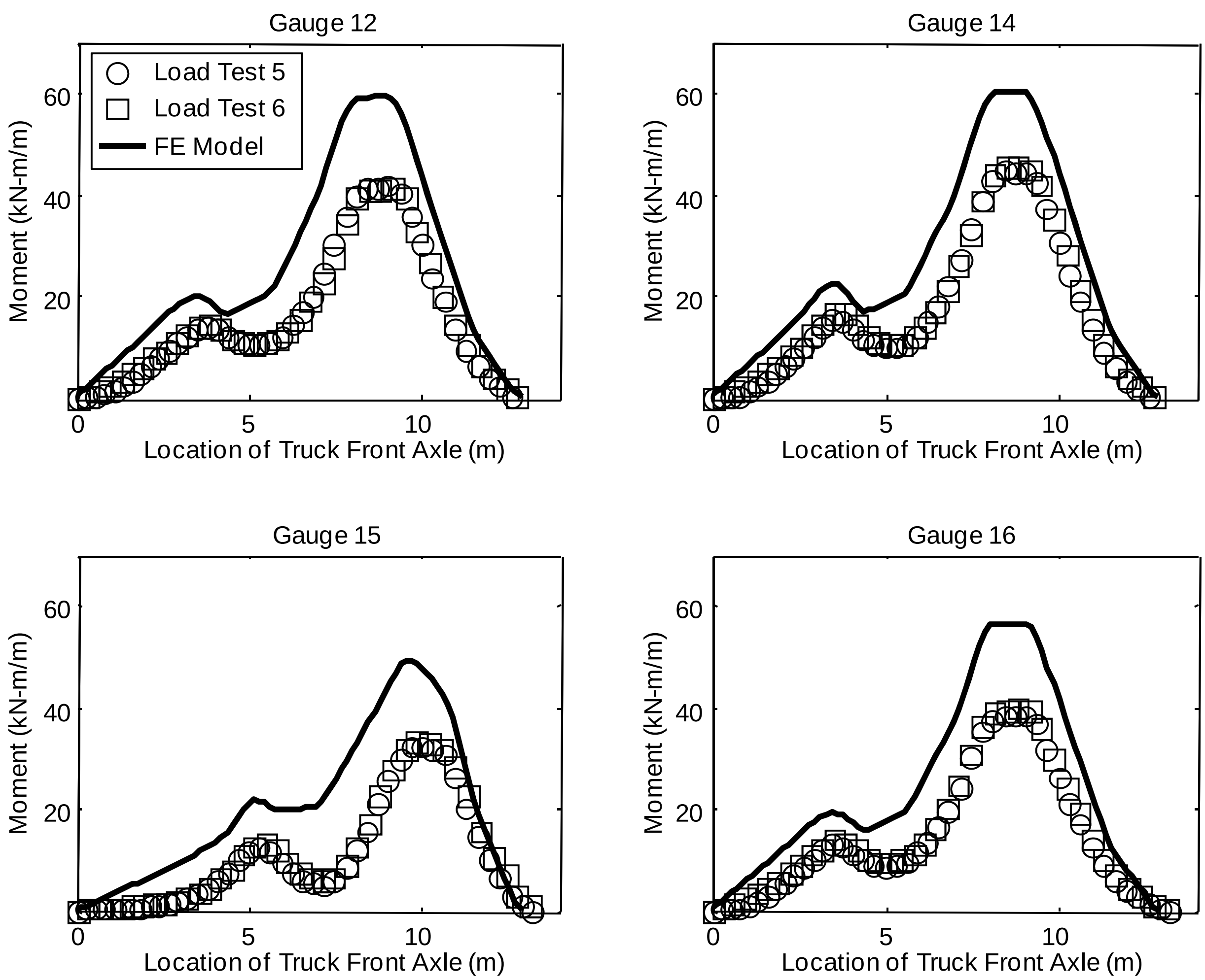

Figure 20 -- Test-derived and FE-predicted slab moments for live load tests 5 and 6

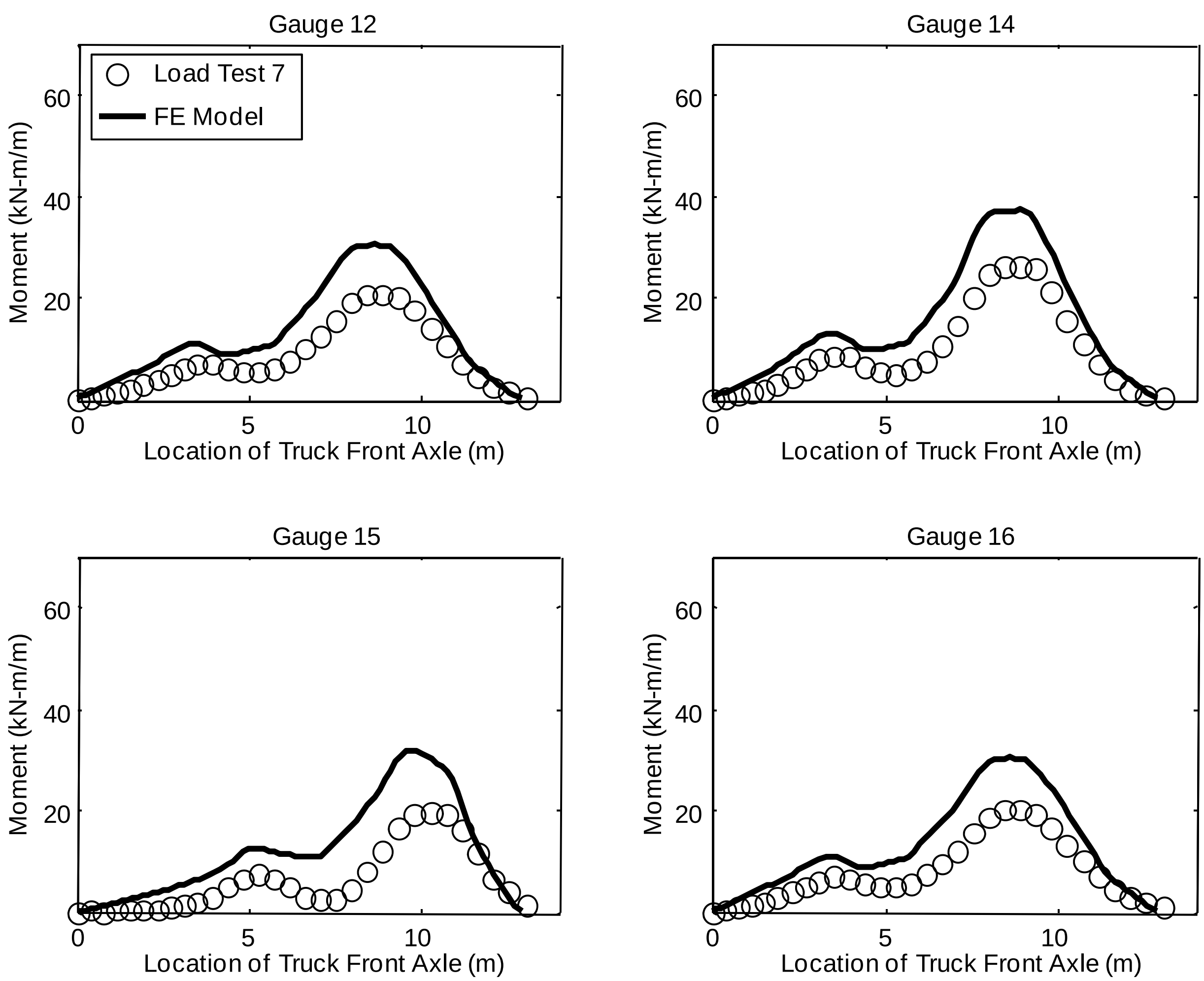

Figure 21 -- Test-derived and FE-predicted slab moments for live load test 7

7.10 Live Load Test Conclusions

The strain measurements were consistent, and the results appear reliable. Measured strains indicate that the bridge is likely un-cracked on the interior part of the bridge (away from the curbs). Comparison of the finite-element analysis results and moments inferred from the live load tests indicate that the FE models are somewhat conservative, and appropriate tools for live load rating flat slab bridges.

8 References

AASHTO (2008). *The Manual for Bridge Condition Evaluation, 1st Ed*. American Association of Highway and Transportation Officials, Washington, DC.

AASHTO (2010). *LRFD Bridge Design Specifications*. American Association of Highway and Transportation Officials, Washington, D.C.

Amer, A., Arockiasamy, M. and Shahawy, M. (1999). "Load distribution of existing solid slab bridges based on field tests." *Journal of Bridge Engineering*, 4(3): 189-193.

Jáuregui, D.V., Licon-Lozano, A. and Kulkarni, K. (2007). "Improved Load Rating of Reinforced Concrete Slab Bridges." Report No. NM05STR-02, New Mexico Dept. of Transportation, Albuquerque, NM.

Poulin, T. (2012). *Development of Improved Analytical Load Rating Procedures for Flat Slab Concrete Bridges*. M.S. Thesis, Dept. of Civil and Environmental Engineering, University of Maine, Orono, ME.

Saraf, V.K. (1998)."Evaluation of existing RC slab bridges." *Journal of Performance of Constructed Facilities*, 12(1): 20-24.

Wight, J.K and MacGregor J.G. (2009). *Reinforced Concrete: Mechanics and Design, 5th Ed*. Pearson Prentice Hall,Upper Saddle River, New Jersey.