FIELD INVESTIGATION OF A STRENGTHENED TIMBER TRESTLE RAILROAD BRIDGE

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ABSTRACT

A three-span, open-deck timber trestle railroad bridge had been previously field load tested. The prior testing program was done in cooperation with the Transportation Technology Center, Inc. a subsidiary of the Association of American Railroads. The bridge was subjected to static and moving train loads as well as well as controlled actuator ramp loading. The bridge was later strengthened by the addition of helper stringers. The bridge was load tested again by moving train loads. Comparisons of the stiffness of the bridge were made before and after the strengthening. The efficiency of the helper stringers was between 82 percent and 97 percent. Load sharing among stringers was determined empirically. No pattern of load sharing among them could be identified, but individual stringers carried up to 9 percent more load share than in an equal load sharing distribution. The transient displacement responses showed predominantly no dynamic impact effect, but isolated increases of 6-10 percent were observed.

EXECUTIVE SUMMARY

Many timber trestle railroad bridges have been in service for 50 to 100 years. Wear and tear on these numerous bridges occurs despite continual maintenance. In some cases severe degradation has been occurring. Consequently, the structural condition of short span railroad bridges is an important national transportation issue. This is particularly a concern on short lines in sparsely populated areas.

The research reported here is the second phase of an examination of rehabilitation needs of existing open-deck timber trestle railroad bridges in the United States. Train carloads and their frequency have increased significantly during the service lives of the existing timber trestle railroad bridges. Indeed, a 30 percent increase in design axle loads is being considered for adoption in the applicable design code. Pursuant to that situation, the Transportation Technology Center, Inc. (TTCI), a subsidiary of the Association of American Railroads, is engaged in a comprehensive effort to examine bridge performance under present day train loads. In a prior field load test program, Colorado Sate University and TTCI researchers examined the structural behavior of three existing timber trestle bridges in a comprehensive field load test program. An MPC report on the outcomes of that phase of the work was published in August 2001.

One of the bridges (a three-span bridge about 40 feet long) was subsequently strengthened by the addition of helper stringers. It was then field load tested under controlled moving train loadings at speeds up to 20 mph. Voluminous displacement data were acquired for the purposes of examining the structural effectiveness of the helper stringers, empirically comparing load sharing with that of the pre-strengthened bridge, and quantifying dynamic impact effects.

The effectiveness of the added stringers was shown to be high, between 82 percent and 97 percent. Load sharing of the post-strengthened bridge was consistent with that finding. Load shares were about as expected due to the added stringers, in comparison with the pre-strengthened bridge. The ideal load share value of 20 percent (25 percent) was exceeded by as much as 9 percent (10 percent) in the post- (pre-) strengthened bridge. Dynamic impact at the 20 mph speed was predominantly negligible, but isolated instances of a 6 percent to 10 percent effect occurred. Thus, no solid recommendation for a non-zero dynamic impact effect can be made for design provisions.

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1. INTRODUCTION

1.1 Background

This report focuses on the second phase of a research project conducted by Colorado State University (CSU) and the Transportation Technology Center, Inc. (TTCI), a subsidiary of the Association of American Railroads (AAR). Field load tests were performed on a strengthened open deck, timber trestle bridge. In the previous phase, field load tests were conducted on the same bridge prior to its strengthening. The aim of this report is to compare the behavior before and after the strengthening.

The first phase was also part of a comprehensive field load test program conducted in 1995 to examine structural behavior of three existing open-deck timber trestle railroad bridges. Testing included static wheel loads applied by a multi-car test train, ramp loads using a track loading vehicle (TLV), rolling wheel load tests using multi-car test trains at various speeds, and exploratory sinusoidal tests at various frequencies using the TLV for dynamic excitation. The goals of the overall study were to assess load paths, empirically assess load sharing within the stringers of the chords, and dynamic impact characteristics.

The AAR and the Mountain-Plains Consortium (MPC) funded the overall study. The MPC support also included analytical studies of the test bridges done by CSU researchers. Structural analysis was performed to augment the empirical assessment of the implications of the test results via various computer models. Technical reports on both phases of the study were prepared by CSU and the TTIC [1-3]. An MPC report was also prepared on the first phase of the study [4]. An M.S. thesis [5] and an M.S. independent study report [6] were prepared on the more extensive implications observed from the analytical modeling. Several papers were presented at and published in the proceedings of various conferences [7-12].

1.2 Objective and Scope

The bridge addressed in this report is located on the west Y, Avondale junction on the Pueblo Army Depot Activity access road to the Federal Railroad Administration's Transportation Technology Center, Inc. near Pueblo, Colorado. It is identified as Bridge No. 101 and is described subsequently. In 1996 it was strengthened by the addition of a helper stringer to the outside of each chord. It was then load tested in 1997. The load test method consisted of a train rolling across the bridge at various desired speeds. Neither static nor ramp load tests were conducted. The objective was to empirically examine the effect of the additional stringers on the stiffness and strength of the bridge. The rolling train tests consisted of passes at speeds ranging from 2 mph up to 20 mph. The bridge was instrumented more extensively for displacement measurements than in the previous field load tests. Unlike the pre-strengthening tests, no relative displacements between components were measured. All member displacement measurements were taken relative to the ground. The response of the bridge was then compared with the pre-strengthening test results. This report provides the primary outcomes in an abbreviated version of the TTIC report [3].

2. DESCRIPTION OF BRIDGE NO. 101

2.1 Original Configuration

Bridge No. 101 is shown in Figures 1 and 2. It is a three-span, open-deck timber railway bridge with a total length of 40.25 feet of typical construction [13]. The individual spans are approximately 13 feet, 14 feet and 13 feet (157.5 inches, 168 inches and 157.5 inches). A walkway was attached on each side of the bridge at tie level. The bridge spans a gap in an embankment.

The main structural components are composed of creosote-treated Douglas fir timbers. The bridge has a slightly curved track (approximately 1-inch chord off-set at center of bridge) supported on 9 foot long railroad ties (8.75 inches wide by 8.5 inches deep). The railroad ties were spaced 17 inches on center. Each chord is composed of four timber stringers separated by about .25 inch gaps. Each stringer is 6.5 inches wide and 15.5 inches deep. Clear span spacing between the two chords is 34 inches. Stringers in each chord were continuous over two spans and placed in the staggered pattern shown in Figure 3. Alternate end-span stringers in each chord are single-span members.

Single-span stringers are 13.1 feet long. Two-span stringers are 27.1 feet long. Near the supports, all four stringers are bolted together. Caps are 13.5 inches wide by 15.25 inches deep and 14 feet long. Six 12-inch (or larger) diameter timber piles support the caps. Piles in the dump bents protrude about 1 foot above ground. Piles in the two intermediate bents protrude about 6 feet above ground.

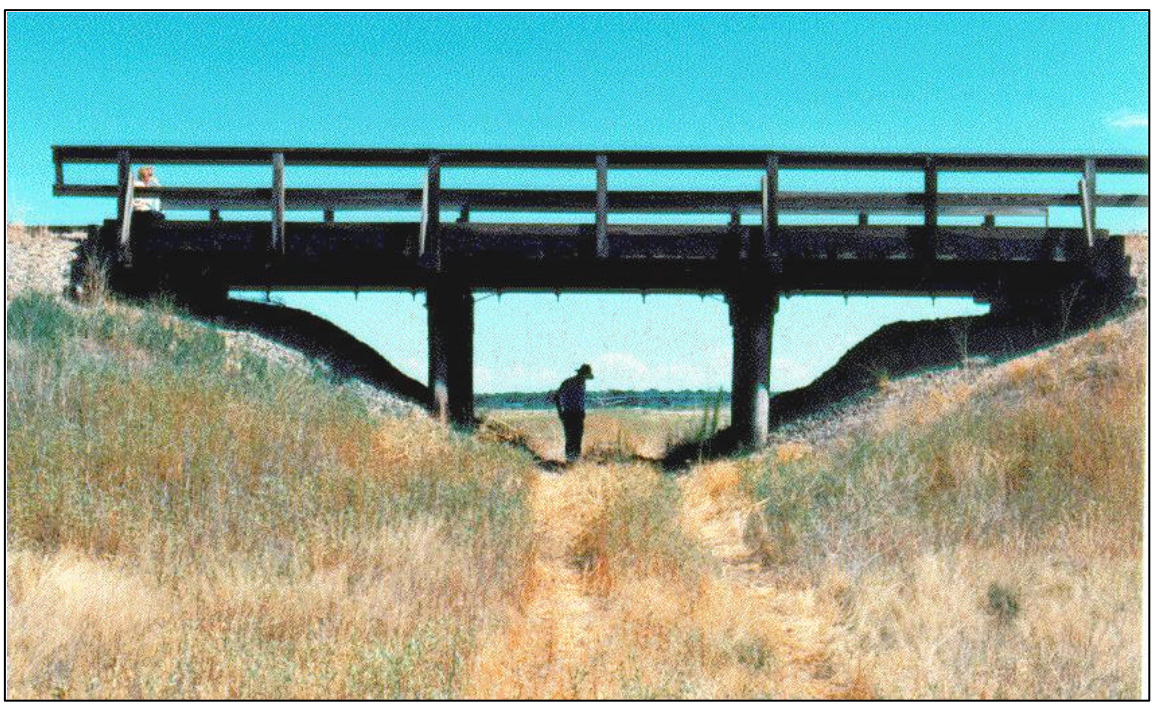


FIGURE 1. Bridge No. 101, View from West Side of Bridge

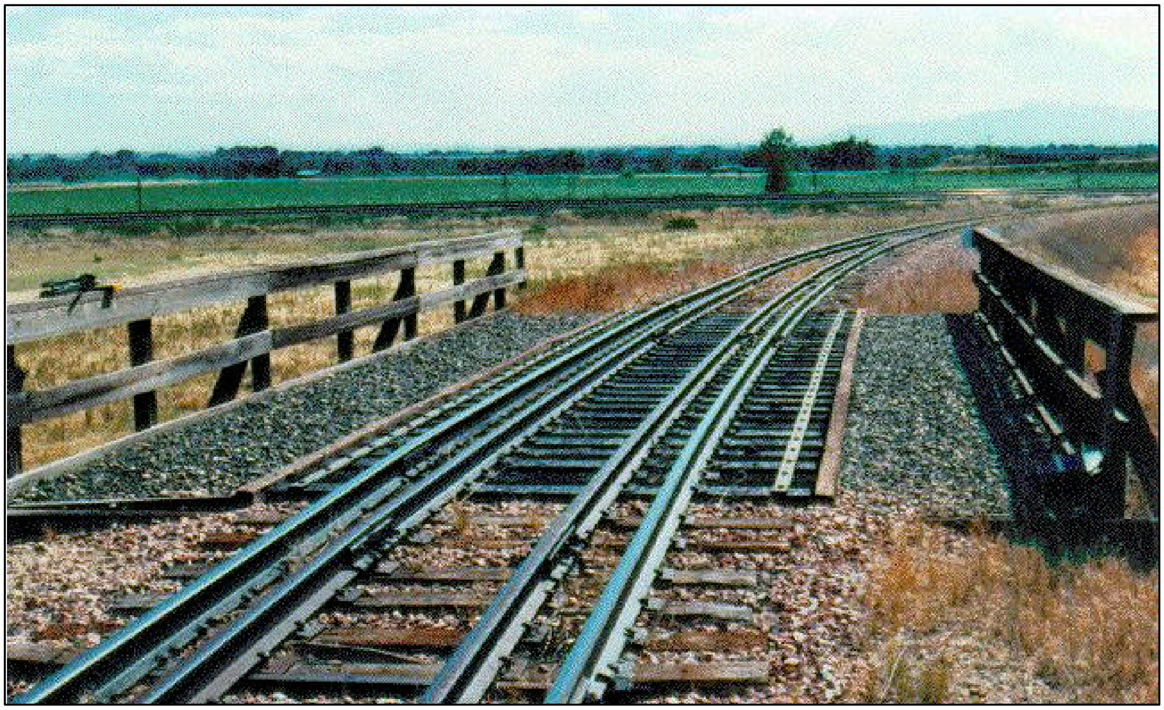


FIGURE 2. Bridge No. 101, View from North End of Bridge

Members were attached with connectors typical to bridge construction. The caps are pinned to the piles and the stringers through-bolted to the caps. The stringers are through-bolted near the caps. Every third tie is through-bolted to the outside stringer of each chord and the tie ends are connected by lag-bolts to a 4-inch by 8-inch guard timber. The walkways cantilever beyond the ends of the caps and are attached to each cap by a strap. It appeared that minor repairs had been made recently, including adding shims on caps, replacing several piles, adding straps between piles and girders, and replacing all deck ties.

2.2 Strengthening of the Bridge

After the initial test of Bridge No. 101, an additional line of exterior stringers was added to each chord. To do this required temporary removal of the two walkways. Lateral bolts connecting the existing stringers were removed and the new helper stringer was inserted between the cap and the wood ties. Two treated stringers were added to each chord, one a two-span member and the other a one-span member. Placement was consistent with the existing pattern of one- and two-span staggered stringers. Stringer sizes and species were the same as the existing stringers. The helper stringers were placed without centering the chord under the steel rail, the easiest and most economical replacement method. Centering would have necessitated removing the rail and disassembly and reassembly of the existing stringers. Longer tie connectors were installed to allow for the added stringer.

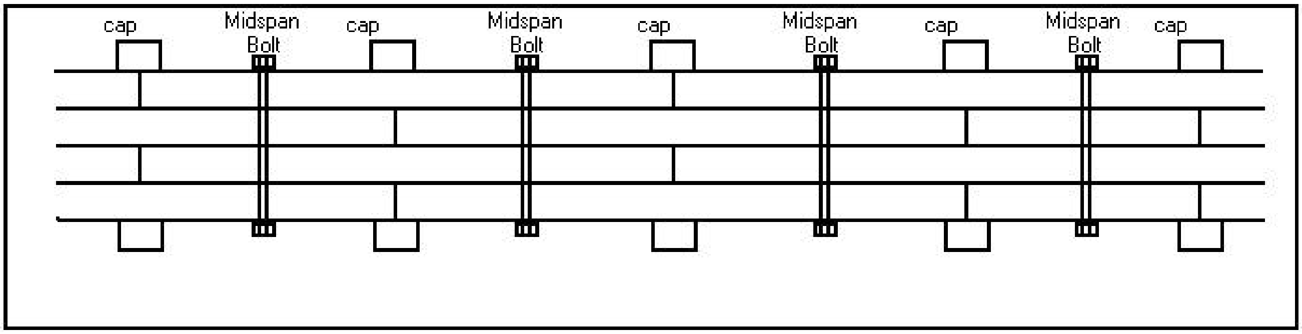


FIGURE 3. Typical placement of Members of Staggered Stringer Construction

3. DESCRIPTION OF THE PRE-STRENGTHENING LOAD TESTS

3.1 Loading Methods

For the first phase of load tests, the TTCI utilized the TLV for loading. The TLV (depicted in Figure 4) is a specialized rail car which provides hydraulic actuator capability by which controlled concentrated loading can be applied to railroad track. Although the TLV is capable of applying load simultaneously in three orthogonal directions, only vertical loading was used in the tests of Bridge No. 101. A pair of side-by-side actuators (one actuator per track line) lift the TLV at its mid-span so as to transfer either part or all of its weight to the track, thus loading the bridge at that location.

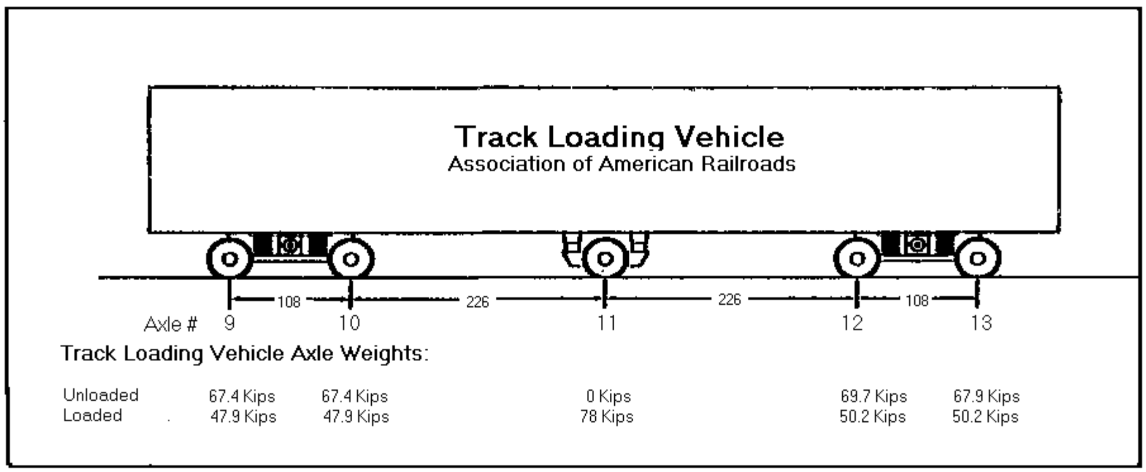


FIGURE 4. Track Loading Vehicle Schematic

Ramp loadings were achieved by positioning the TLV at selected locations along the bridge. The actuators were used to apply a controlled, incremented concentrated loading and unloading at each location. Locations consisted of the ends and mid-span of each span at the ends and other intermediate points of interest. In total, ramp loadings were conducted at 42 locations. Measurements were first taken at a zero load level and then at specific applied load increments achieved by controlling the TLV actuators. Typical axle load levels at the car body location were 0, 30, 60 and 78 kips.

In the test configuration, an instrumentation car (IC) and a locomotive are included in tandem with the TLV. A four-axle locomotive was used for the pre-strengthening tests of Bridge No. 101. A photo of the test train is shown in Figure 5 and its configuration is shown in Figure 6. The axle weights are listed in Table 1. Static loading cases were achieved by positioning the three-car test train (12 axles) at specific locations with reference to the bridge being tested. For a given loading sequence, the train was positioned just off the bridge and instrumentation measurements were taken (instrumentation was "zeroed"). Then the train was slowly moved and stopped at the first predetermined position and measurements (electronic data and optical back-up data) were taken again. After that, the train was moved to the next position and the measurement process was repeated, then moved again to next position of interest, etc. A total of 108 positions were used and are described in the previous reports [1, 2, 3].

FIGURE 5. Test Train Composed of Locomotive, Instrumentation Car and Track Loading Vehicle at Bridge 101.

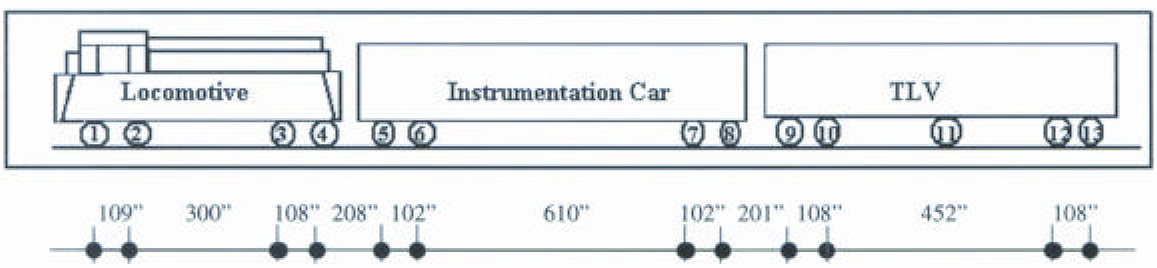


Figure 6. Test train axle spacing pre-strengthening tests

TABLE 1. Train Axle Weights (August 1995)				
	Front Truck Weight, (lb) Axle A	**Front Truck Weight, (lb) Axle B**	**Rear Truck Weight, (lb) Axle A**	**Rear Truck Weight, (lb) Axle B**
Locomotive	67,500	67,500	67,500	67,500
Instrumentation Car	33,675	33,675	32,475	32,475
TLV	67,400	67,400	69,675	69,675

For reference purposes, recording the location of a selected axle of the train system identified each position of the train. The "positioning" of an axle implies specifying at what location along the bridge the selected axle was to be located in order to achieve the desired load effect of the entire test train. For example, if axle 4 was to be positioned at a location B (the letter "B" indicating either the mid-span of a particular span or a point directly on top of a selected pier or a point adjacent to a selected pile bent), the position identifier used was "4@B". In other load cases, two axles were centered about a selected point on the bridge. An example situation was to position two closely spaced loads to be equidistant from mid-span of a particular span. Another case of interest was to center loads about a pier to maximize the reaction at that location. Thus, as an example, the identifier "4-5@D" was used to indicate centering axles 4 and 5 about point D.

Rolling trainload tests were conducted by recording the electronic data while the test train passed over the bridge. The engineer controlled speed until approaching the bridge and then allowed the train to continue unassisted. The speed of the train on the bridge was estimated by using a stopwatch to time the train passing between two given points on the bridge.

3.2 Instrumentation

Details of the instrumentation used are provided in the past report and thesis [1,5] and are briefly described herein. A combination of displacement transducers and extensometers were used. Linearly variable displacement transducers (LVDTs) were used to measure vertical displacements. The LVDTs were installed to measure relative movement between components and also with reference to the ground, depending on the deflection data sought. The extensometers were used to measure longitudinal deformation in selected members. LVDTs were repositioned for various loading sequences to acquire different desired data. Data was either collected by a single scan of all data acquisition channels triggered by command, as in the case of static and ramp testing, or by continuous scanning of the channels as used in the rolling vehicle tests. For static and ramp load testing the deflection instrumentation was backed up with optical survey equipment.

4. DESCRIPTION OF THE POST-STRENGTHENING LOAD TESTS

4.1 Loading Methods

After the retrofit, Bridge No. 101 was tested in March 1997 using a locomotive and three boxcars. The configuration of the train is shown in Figure 7. Table 2 lists the axle weights. Only rolling train loadings were used. The train made six passes across the bridge for each of the five speeds - 2, 5, 10, 15, and 20 mph.

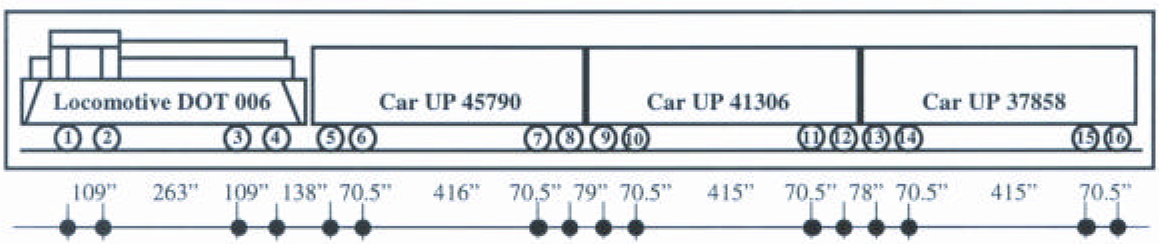


Figure 7. Test train axle spacing post-strengthening tests

TABLE 2. Train Axle Weights (March 1997)

	Front Truck Weight, (lb) Axle A	Front Truck Weight, (lb) Axle B	Rear Truck Weight, (lb) Axle A	Rear Truck Weight, (lb) Axle B
DOT 006	59,677	59,677	60,438	60,438
UP 45790	70,816	70,816	70,343	70,343
UP 41306	65,403	65,403	67,017	67,017
UP 37858	66,811	66,811	65,581	65,581

4.2 Instrumentation

String potentiometers were installed to measured vertical displacements at 72 locations on the bridge. TTCI personnel instrumented and monitored 60 locations; CSU personnel instrumented and monitored 12 locations. All displacements were measured relative to the ground.

The instrumentation locations and data referencing nomenclature are shown in Figure 8. Within any span, five instrumentation positions (Positions #1 to #5) representing the sixth points of a span are referenced, but not every stringer was monitored at each point. The longitudinal stringers are assigned numbers 1 through 10. The letters A, D, G, and J, going from south to north, references the four cap beams. Points B, C, E, F, etc. are intermediate locations along the spans. The letter references are the same as were assigned during the pre-strengthening load test.

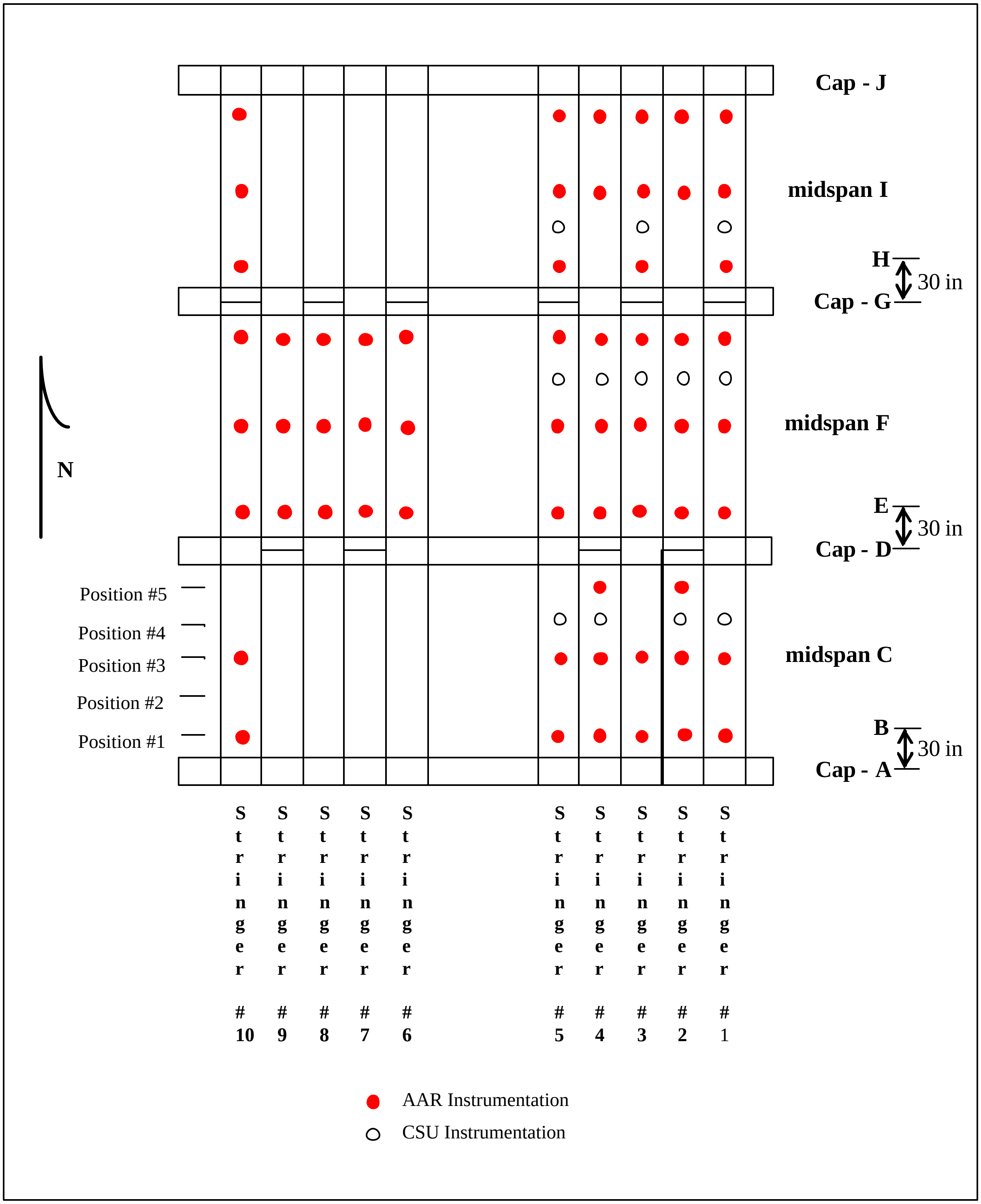


FIGURE 8. Bridge No. 101 - Plan View

5. MATERIAL PROPERTIES

The wood material was visually identified as Douglas fir, most likely treated to a high retention level with an oil-based preservative treatment (generically known as “Penta”). The helper stringers were apparently the same. Modulus of elasticity (MOE) of the all stringers, cap and selected pile members was measured by an ultrasonic-based, non-destructive assessment technique [14]. The procedure and a detailed assessment results are described in the earlier reports and thesis [1,2,4]. Individual MOE values measured for each stringer are displayed in Fig. 8. Values shown are actual digital readings from the instrument used. Rounding to four significant figures, the MOE values range between 1,325,000 psi and 2,194,000 psi. The average value is 1,775,000 psi with a standard deviation of 248,500 psi. The mean value for the existing (added) stringers is 1,762,000 psi (1,825,000 psi), with a standard deviation of 268,500 psi (149,000 psi). Considering the bridge has a long service in dry climate, these results are deemed reasonable.

6. RESULTS OF THE PRE-STRENGHTENING LOAD TESTS

Comprehensive load test results for response of Bridge 101 before strengthening are available in prior reports and thesis [1-4]. Selected results are presented herein as they are pertinent to comparisons made with the response of the strengthened bridge.

6.1 Static Loads

For the pre-strengthened bridge tests, four static load positions were identified as suitable for making the comparison of displacement responses with the corresponding quasi-static responses of the post-strengthened bridge. Using the identifiers of the pre-strengthened bridge test, the positions are Load 1@H, Load 1@F, Load 1@E and Load 1@B, respectively. These constitute four stops in the sequence of static load positions used as the test train was moved from south to north across the bridge. For these load cases, data were taken for the two end-spans only. Displacement data (measured relative to the ground) for each of these selected static loadings are presented in Figures 9 through 12. The numbering of the eight stringers is from the east side to the west side of the bridge. Figure 9 shows data for Load 1@H, i.e. when the front two axles (truck) of the locomotive were centered at or about the mid-span of Span 1. Figure 10 shows data for Load 1@F, i.e. when the front axle of the locomotive was centered at or about the mid-span of Span 2. Figure 11 shows data for Load 1@E, i.e. when the front two axles of the locomotive were centered at or about the mid-span of Span 2. Figure 12 shows data for Load 1@B, i.e. when the front two axles of the locomotive were centered at or about the mid-span of Span 3.

6.2 Moving Loads

The transient response was measured for each pass of the moving test train. The prior report [1] contains extensive assessment of the resulting data. Pertinent results are used subsequently in this report.

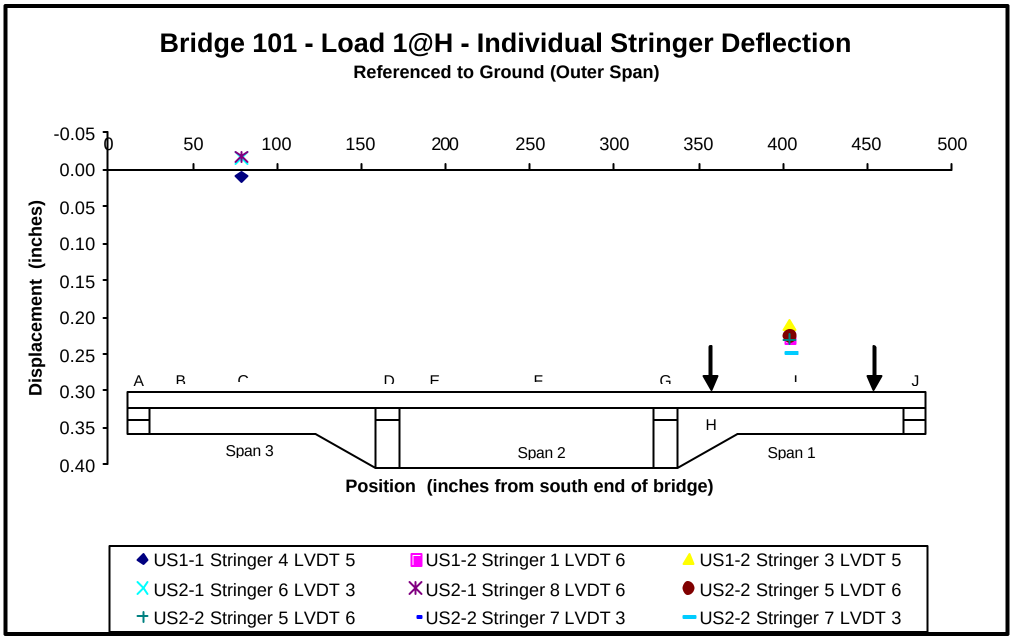


Figure 9. Transducer deflection measurements referenced to ground of Bridge 101 for load 1@H.

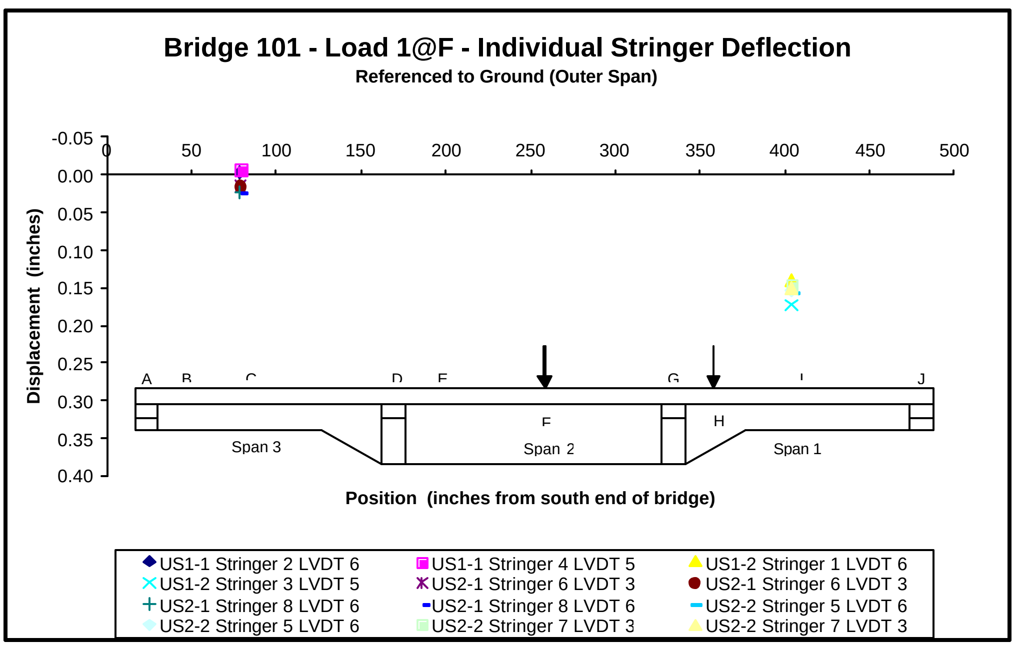


Figure 10. Transducer deflection measurements referenced to ground of Bridge 101 for load 1@F.

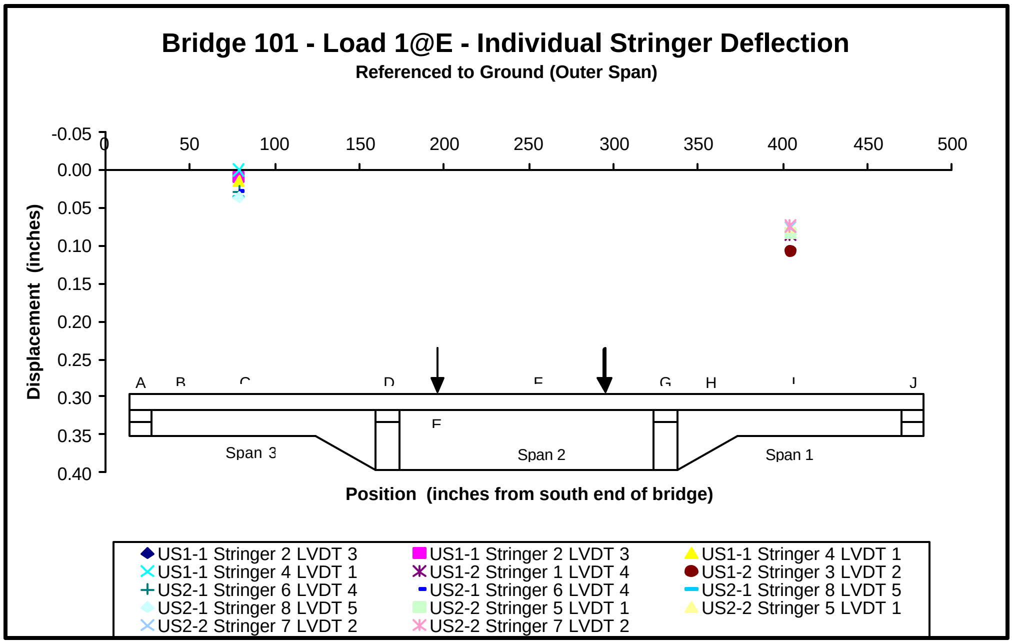


Figure 11. Transducer deflection measurements referenced to ground of Bridge 101 for load 1@E.

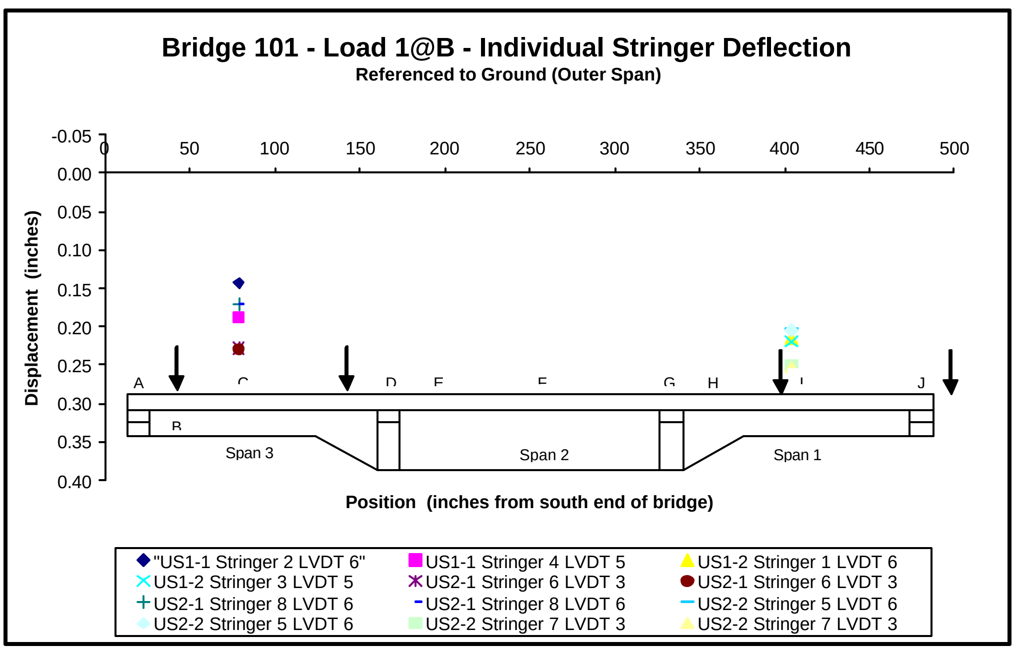


Figure 12. Transducer deflection measurements referenced to ground of Bridge 101 for load 1@B.

7. RESULTS OF THE POST-STRENGTHENING LOAD TESTS

7.1 Quasi-Static Loads

The response of the bridge was measured for a range of speeds between 2 mph and 20 mph. As the observed differences between the responses for the various speeds were small, only data for the 2 mph and 20 mph speeds are described herein. Since the 2 mph data are only moderately different than the 20 mph data, the data for the 2 mph speed are used herein in as the "quasi-static" load responses. In other words, the 2 mph results are considered acceptable for representing the response at a 0 mph train speed in the manner described below.

Displacement data for the 2 mph and 20 mph speeds of the train was taken for three passes at each speed. To be able to extract the displacement data for a particular train position, the corresponding time was needed. Inductive triggers were located directly above both interior bents and two feet outside the end bents. The times representing the train positioned over the two adjacent bents were available from the trigger data. This time corresponding to each particular intermediate train position of interest was calculated by linearly proportioning the two times corresponding to the train passing over the two adjacent bents.

Four instantaneous positions of the moving train that constitute load positions that approximately correspond to the four selected static load positions examined in the pre-strengthening load tests were of interest. Each of these was examined at the 2 mph and 20 mph train speeds. The 2 mph results were compared with the 20 mph results to examine dynamic impact effects. The 2 mph results were also used for comparison with the pre-strengthening static load test results.

For reference purposes, the quasi-static load cases are identified by the nomenclature “Load Case n@v mph” where n is a load case number and v is the train speed. Load Case 1@2 mph and Load Case 1@20 mph have the front axle of the moving locomotive centered at or about the mid-span of Span 1 of the bridge, but moving at 2 mph and 20 mph, respectively. This position is chosen as it reasonably corresponds to static Load 1@H (see Figure 9) in the pre-strengthening load tests. Load Case 2@2 mph and Load Case 2@20 mph have the front pair of axles of the moving locomotive centered at or about the mid-span of Span 2. These load cases are chosen as reasonably corresponding to static Load 1@E (see Figure 11) pre-strengthening load tests. Load Case 3@2 mph and Load Case 3@20 mph had the front pair of axles of the moving locomotive centered about the mid-span of Span 3. They were chosen as reasonably corresponding to static Load 1@B (see Figure 12) in the pre-strengthening load tests. Load Case 4@2 mph and Load Case 4@20 mph had the first axle of the moving locomotive centered at or about Span 2. They were chosen as reasonably corresponding to static Load 1@F (see Figure 10) in the pre-strengthening load tests. For ease and presentation purposes, the pre-strengthening loadings 1@H, 1@E, 1@B, and 1@F are subsequently called Load Cases 1@0 mph, 2@0 mph, 3@0 mph and 4@0 mph, respectively.

Displacement data for all the quasi-load cases were obtained for each of the three passes of the train and mean values were determined. These mean values provide the basis for the various graphical plots and tables presented within the body of this report.

Figure 13 shows some collective results for the quasi-static loadings. The mean mid-span displacements values for the stringers (average of the individual stringer mid-span displacement values within a span for all passes combined) along the bridge are plotted for each of Load Cases 1@2 mph, 2@2 mph, 3@2 mph, and 4@2 mph. The displaced various shaped curves are qualitatively consistent with the corresponding loadings but also give evidence of small support settlements at two intermediate bents, i.e. at cap D and cap G.

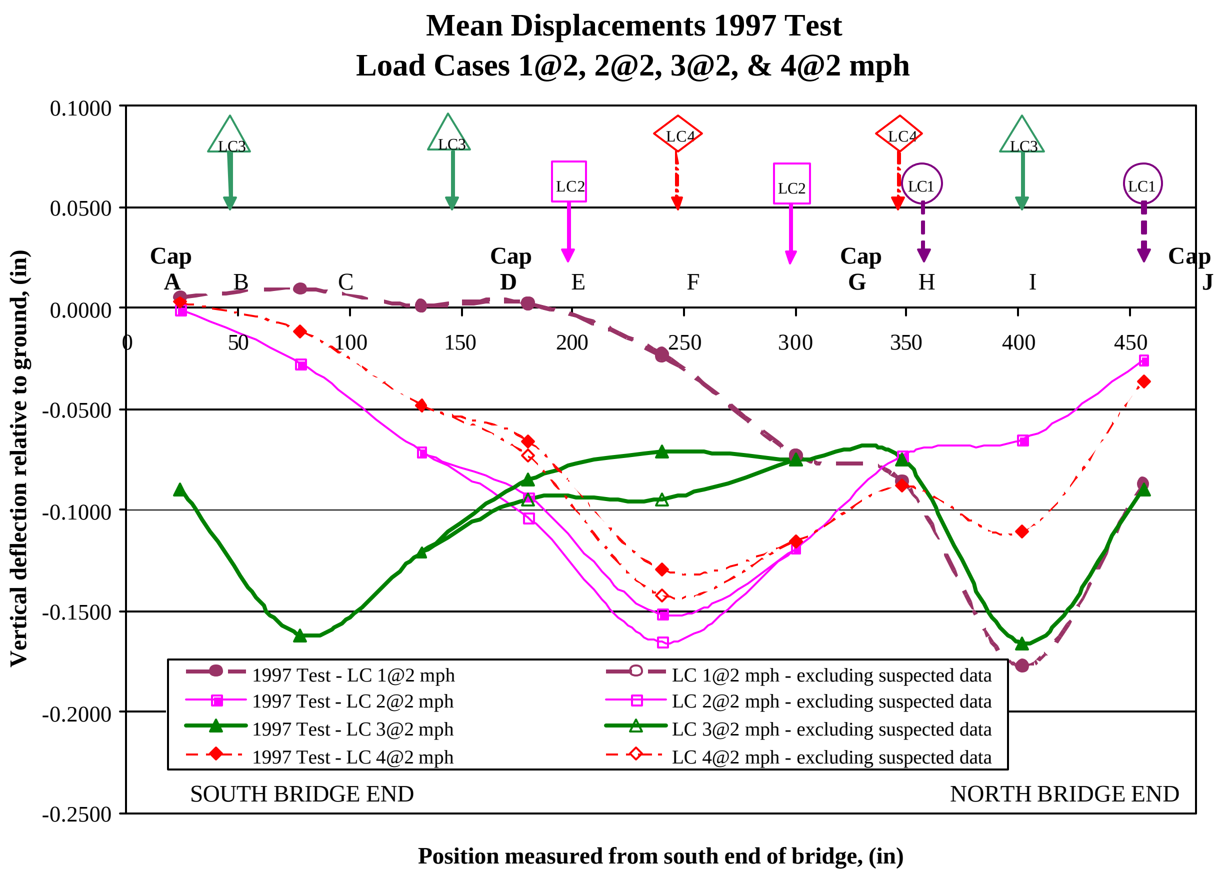


FIGURE 13. Mean Displacements Post-Strengthening Load Cases 1@2, 2@2, 3@2, and 4@2 mph.

7.2 Consistency of Data

Prior to an in depth analysis of the data collected during the post-strengthening load test, a check was made to confirm that the data were consistent between two runs of the same nominal speed. Details are available in the TTCI report [2]. This check was performed on displacement data resulting from two passes at a nominal speed of 2 mph and two passes at a nominal speed of 20 mph. In the evaluation of the data, it was observed that the potentiometer that measured the displacement of stringer 4 in Span 2 probably malfunctioned. Specifically, that stringer was generally recorded as having distinctly lower measured displacement values than the adjacent stringers, for all train passes at 2 mph. Thus, where applicable, data reported subsequently are presented both with and without that potentiometer recording included. Stringer 4, Span 2 data did not appear to be suspect at the 20 mph speed. This is possible because electronic connections etc. were checked in between the various train speed increments and the cause may have been corrected as a result.

Calculated coefficient of variation (COV) values for the measured displacements (means of the stringer values) were obtained for the three passes of the train at 2 mph and 20 mph, respectively. All COV values were low,

mostly ranging between 0.5 percent and 6.1percent, with the exception of Span 2. In Load Cases 1@2 mph and 3@2 mph, data for that span had a COV of 41 percent. The displacement value for train Pass #3 appeared to be a suspect value. With that value excluded, the coefficient of variation for Span 2 was 9.1 percent. Values obtained with stringer 4 data for Span 2 excluded but including Pass #3 is 44.0 percent. With both stringer 4 and Pass #3 excluded, the value is 7.5 percent. For the other quasi-static load cases, it was 12.9 percent to 22.2 percent
.

7.3 Moving Train Loads

Transient response to each pass of the test train was measured at all instrumentation positions Representative results are presented and discussed in Section 9.0.

8. COMPARISON BETWEEN PRE- AND POST-STRENGTHENING RESULTS

Differences in the test trains used in the pre- and post-strengthening load tests complicate a comparison of results. Thus, adjustments to the data are made to reflect the differences. As stated earlier, as no static load tests were conducted on the post-strengthening tests; instances in time of movement of the train are used subsequently to produce quasi-static loadings. Displacement results for the pre-strengthening load test were used to calculate the anticipated displacement response of the bridge after the retrofit. These values were compared with the measured results for the pseudo-static load case at 2 mph. To make an equivalent comparison, the pre-strengthening displacements were "adjusted" in two steps to account for: a) the difference in loads and b) the difference in the number of stringers of the original and retrofit bridge conditions. Raw data displacement values were scaled by the ratio of the two different loadings (sum of the axle loads on the span of interest) to obtain the "load-adjusted" pre-strengthening displacement values. This allows one to approximately compare the displacements of the bridge as if it were subjected to the same loading before and after retrofit. The load-adjusted pre-strengthening displacement values were then decreased by a factor of 4/5 to account for the additional stringer added in each chord. These "load and stringer adjusted" (or, "fully adjusted") pre-strengthening displacements constitute an estimate of what the post-strengthening measured displacement values would be after the retrofit, if the added stringers were fully effective.

For the pre-strengthening load tests, instrumentation was moved about and was not in place for all spans for all loadings. Consequently, for the quasi-static load cases no past comparative data existed for Span 2. Indeed, only quasi-static Load Case 1@2 mph (corresponding to pre-strengthening static loading 1@H) and quasi-static Load Case 3@2 mph (corresponding to pre-strengthening static loading 1@B) had data needed to apply to the comparison.

In Tables 3 and 4, the mid-span mean deflections in 1997 post-strengthening Load Cases 1@2 mph and 3@2 mph are compared with those of the 1995 pre-strengthening Load Cases 1@0 mph (1@H) and 3@0 mph (1@B), respectively. In absolute terms (actual measured values), the mean mid-span deflections under loadings in the post-strengthening test ("1997 Test" in the table) were much lower than those under the loadings in pre-strengthening test ("1995 Test" in the table). The load ratio used to adjust the pre-strengthening displacements was (59,677 lbs/67,500 lbs) except for Load Case 3@0 mph at Span 1. Due to the third axle load being on that particular span in Load Case 3@0, that particular load ratio was (60,438 lbs/67,500 lbs). As expected, the load-adjusted pre-strengthening mean mid-span downward displacements are still greater than those in post-strengthening tests. This simply indicates that the bridge is stiffer after the retrofit than before it, as expected.

TABLE 3. Midspan Mean Displacements Resulting from 1997 Test Compared to 1995 Static Test for Load Case 1 (1@H)

Load Case 1 (1@H)	Midspan Displacements (in)			
		1995 Static Test		1997 Test
	Span	Original load P = 67,500 lbs	Adjusted by 59,677 / 67,500	Load on span P = 59,677 lbs
	Span 1 (G-J)	0.2321 down	0.2052 down	0.1771 down
	Span 3 (A-D)	0.0077 up	0.006808 up	0.0094 up

TABLE 4. Midspan Mean Displacements Resulting from 1997 Test Compared to 1995 Static Test for Load Case 3 (1@B)				
	Midspan Displacements (in)			
	Span	**1995 Static Test**		**1997 Test**
Load Case 3 (1@B)	**Span 1 (G-J)**	Original load P = 67,500 lbs	Adjusted by 60,438 / 67,500	Load on span P = 60,438 lbs
		0.2243 down	0.2008 down	0.1659 down
	Span 3 (A-D)	Original load P = 67,500 lbs	Adjusted by 59,677 / 67,500	Load on span P = 59,677 lbs
		0.1885 down	0.1667 down	0.1618 down

Table 5 provides a further assessment of the effectiveness of the retrofit as determined by the additional adjustment for the number of stringers. The third column contains the pre-strengthening test values of the mid-span mean deflections after being scaled by load ratios. These values were then decreased by the factor of 4/5 to obtain the values tabulated in the fourth column. The efficiency (effectiveness) of the retrofit was determined by dividing the fully adjusted mean mid-span displacements by the corresponding absolute measured mean mid-span displacements from the pre-strengthened load tests. For Load Cases 1@2 mph and 3@2 mph, the calculated efficiencies of Span 1 are 92.7 percent and 96.8 percent, respectively. For Load Case 3@2 mph the calculated efficiency of Span 3 (Span A-D) was 82.4 percent. The average efficiency of the bridge retrofit is 91 percent. If the added stringers provided stiffness equal to the original stringers the efficiency would approach 100 percent.

TABLE 5. Percent of Efficiency of Retrofit with an Additional Ply in 1997 Resulting from 1997 Test Compared to 1995 Static Test					
Load case	**Midspan Displacements (in) as if under the same loads in 1997 test**				**Efficiency**
	Span	**1995 Static Test**		**1997 Test**	
		4 plies before retrofit	**5 plies theoretically**	**5 plies after retrofit**	
LC1 (1@H)	**Span 1 (G-J)**	0.2052 down	0.1642 down	0.1771 down	92.7 %
	Span 3 (A-D)	0.006808 up	0.005446 up	0.009400 up	more uplift
LC 3 (1@B)	**Span 1 (G-J)**	0.2008 down	0.1606 down	0.1659 down	96.8 %
	Span 3 (A-D)	0.1667 down	0.1334 down	0.1618 down	82.4 %

Figures 14-17 graphically compare the post strengthening displacement responses for the 2 mph vs. 20 mph train speeds. (Figure 14 (15, 16, 17) is for Load Cases 1@2 mph vs. 1@20 mph (2@2 mph vs. 2@20 mph, 3@2 mph vs. 3@20 mph, 4@2 mph vs. 4@20 mph). The responses of the bridge to the moving train at the nominal 2 mph and 20 mph speeds are very close to each other. Note that the displacement response at 20 mph was predominantly less than that at 2 mph. The inference is that the dynamic impact effect was essentially non-existent.

The results listed in Table 5 are displayed graphically in Figures 18 and 19. The mean displaced shape for Load Case 1@2 mph is illustrated in Figure 18. The plot is done both including and excluding the suspect data for stringer 4 in Span 2, with no noticeable difference. At two locations (C and I) the actual (measured) mean pre-strengthening displacements for Load Case 1@0 mph (Load 1@H) are plotted. These values are also plotted after: a) adjusting for the difference in load level for the two test trains and then b) adjusting for the additional stringer. The resulting value is close to the plot for the post-strengthening displaced shape. Companion information for Load Case 3@2 mph vs. Load Case 3@0 mph (Load 1@B) is shown in Figure 19. Excluding the suspect stringer 4 data in Span 2 makes a noticeable difference within Span 2 only. The result underscores that stringer 4 data is suspect as the adjusted shape has curvature that better matches the sense of the loads. In particular, Span 2, which has no loads on it, should likely have concave downward curvature.

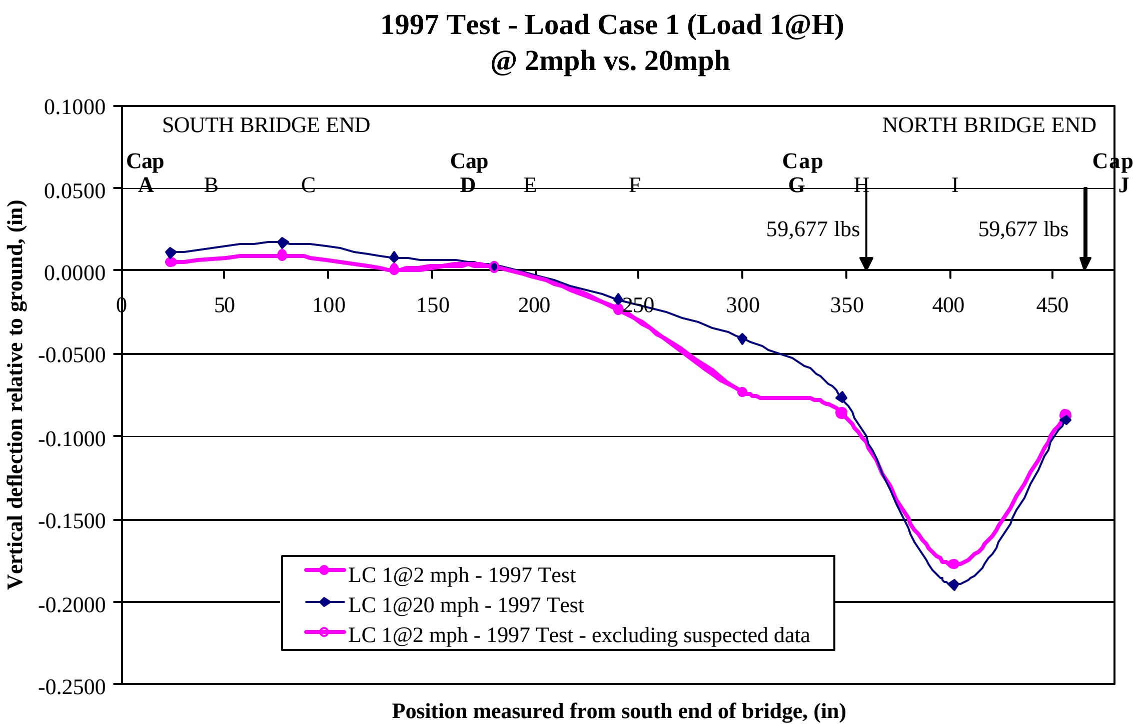


FIGURE 14. Load Case 1- Post-Strengthening Test @ 2 mph and 20 mph

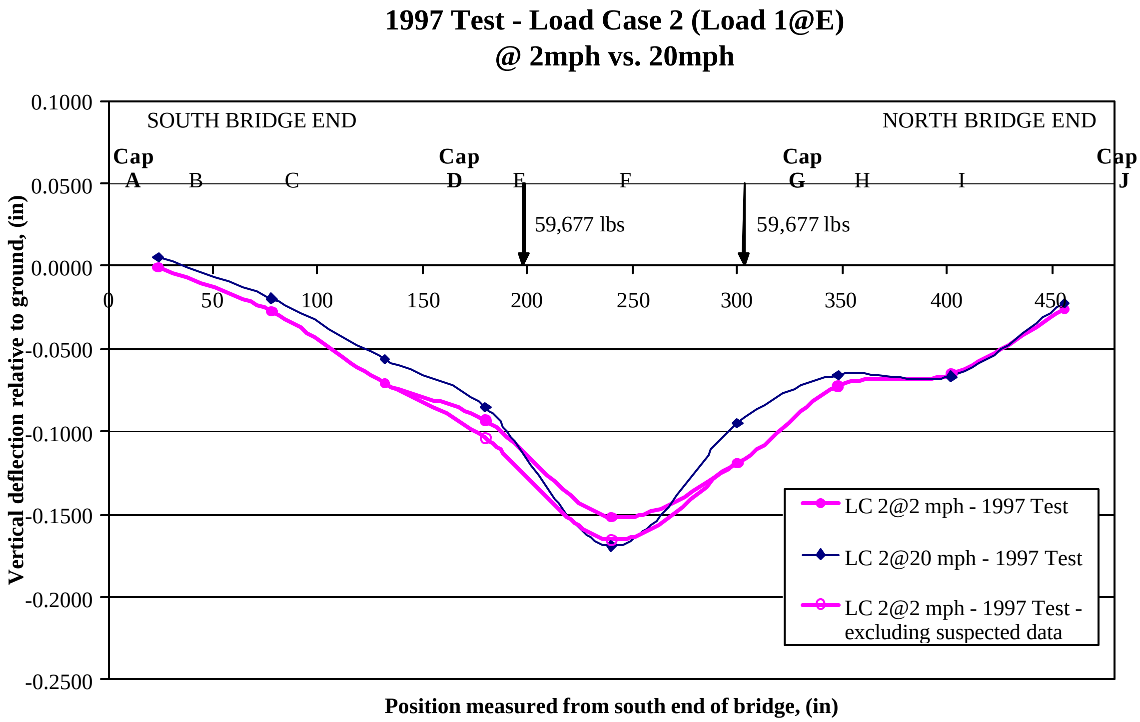


FIGURE 15. Load Case 2- Post-Strengthening Ttest @ 2 mph and 20 mph

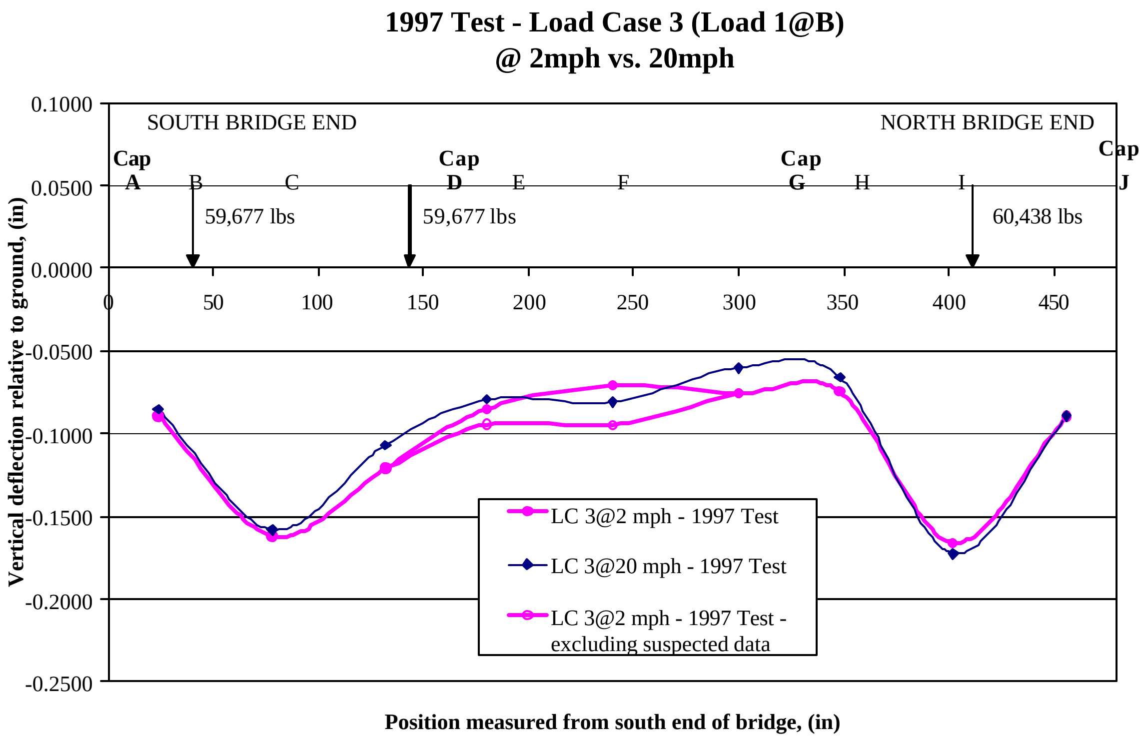


FIGURE 16. Load Case 3 - Post-Strengthening Test @ 2 mph and 20 mph

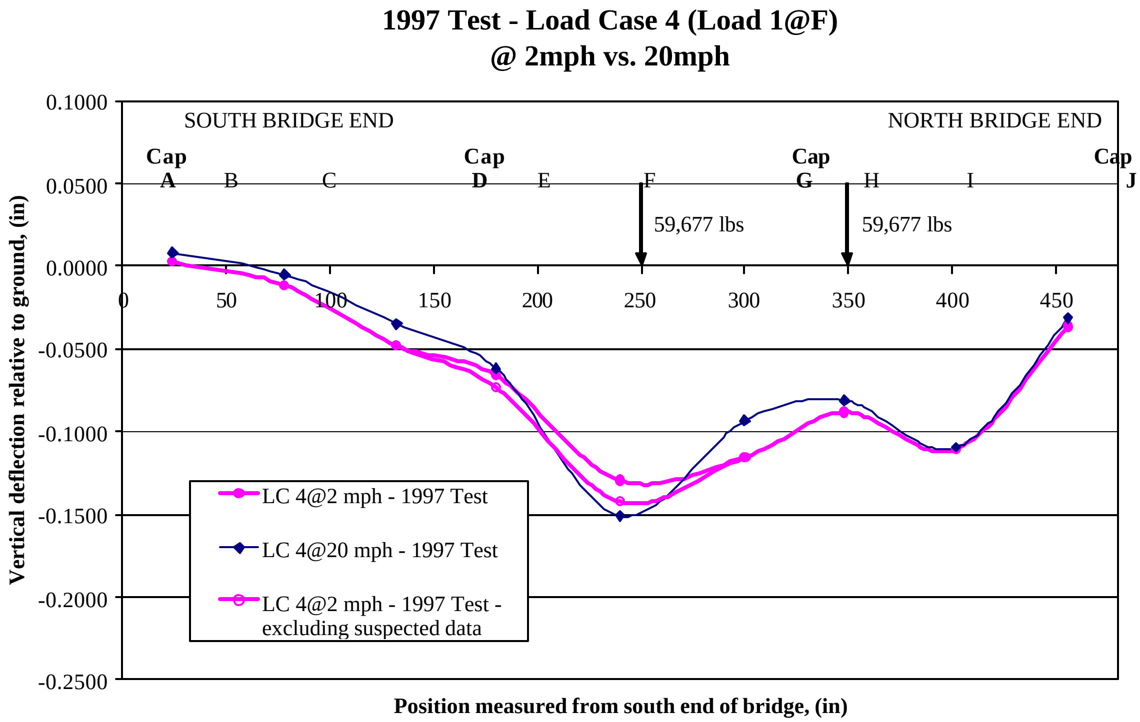


FIGURE 17. Load Case 4 Post-Strengthening Test @ 2 mph and 20 mph

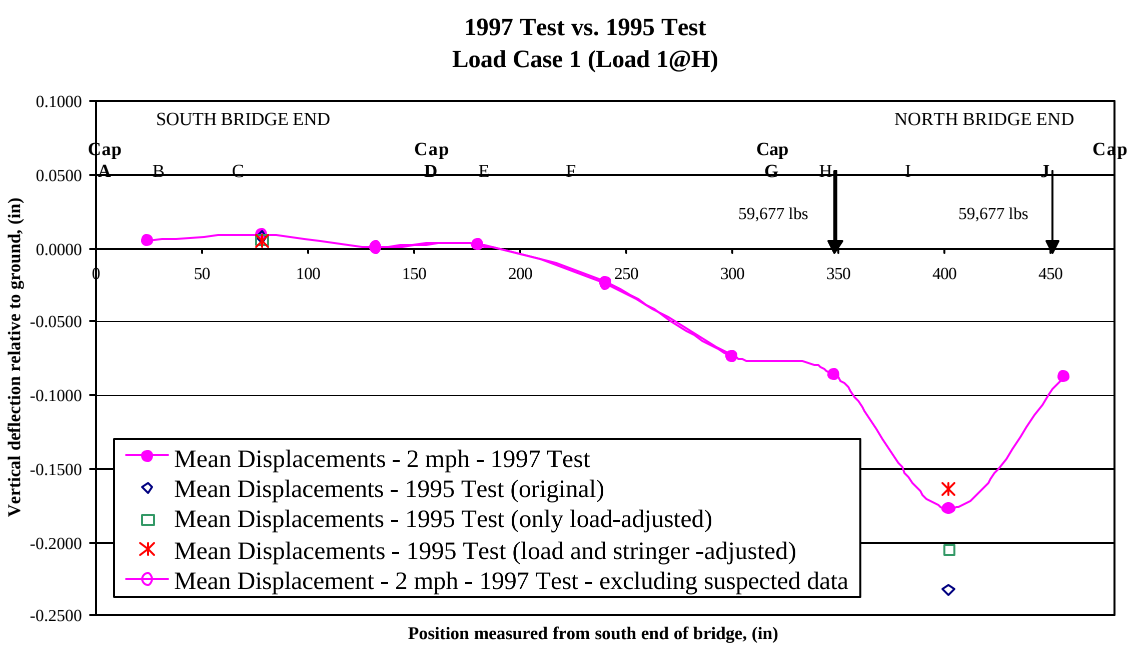


FIGURE 18. Load Case 1 (Load 1 @ H) Post-Strengthening Test vs. Pre-Strengthening Test

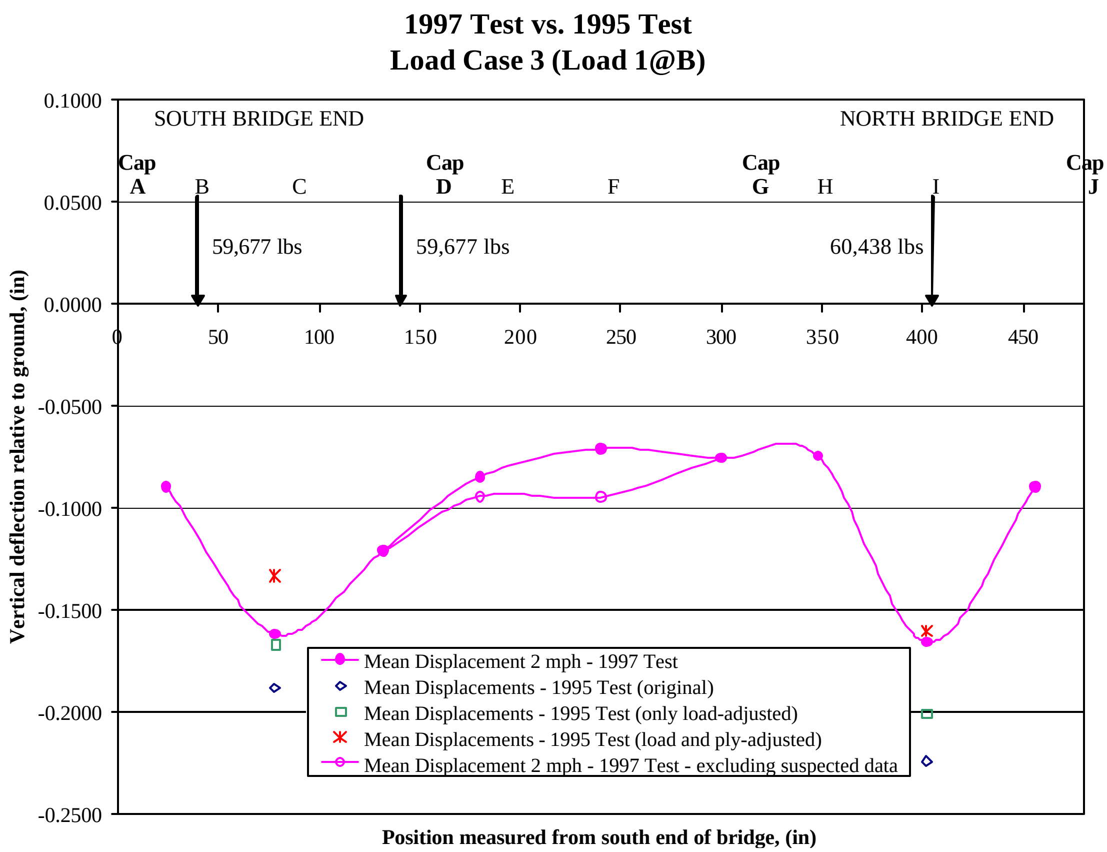


FIGURE 19. Load Case 3 (Load 1 @ B) Post-Strengthening Test vs. Pre-Strengthening Test

9. METHOD TO CALCULATE LOAD SHARING

Since the concentrated applied loads are involved, it is rational to consider that load share in a stringer is a function of EI?/L^3. Each ply of a chord has the same nominal length, L, and moment of inertia, I, values. Thus, relative E (MOE) values are an indicator of relative stiffness of the stringer. On that basis, load share can be approximated by examining relative values of the product of ? (measured displacement) times MOE (measured E) for the stringers within a given chord and span. However, for end-spans the difference in continuity of the stringer is an additional factor. In the end-span chord, the stringers alternate between simply supported, single-span members and two-span continuous members. Resistance to deflection (stiffness) differs due to support condition. An adjustment can be made to approximately account for this difference.

If the stringer is continuous over two spans, with one span loaded at mid-span by a concentrated load, the displacement under the load is:

$$? = (PL^3/48EI) - (3PL^3/512EI) = .0149\ PL^3/EI$$

For a simply supported stringer loaded at mid-span by a concentrated load, the mid-span displacement is:

$$? = PL^3/48EI = .0208\ PL^{3/}EI$$

To approximately account for the relative stiffness of stringers, the correction factor CF = 0.0208/0.0149 = 1.40 is used to adjust the displacement of the two-span continuous ply before it to the value for a simply supported ply. In effect, the CF for the simply supported stringers is 1.0. This adjustment is in addition to that reflecting the difference in MOE values.

The load share is determined by calculating the product of CF x MOE x ? ?for all stringers in a given chord and span, summing the values and dividing each individual value by the total. This yields the proportion of total load shared by each stringer. Multiplying by 100 gives the percent of load shared by each stringer.

The above considerations do not account for support motions that might occur, i.e., they presume the end supports are vertically rigid. To obviate this problem, one should use data measured relative to the “displaced chord” (meaning the line connecting the displaced ends of the stringer).

10. LOAD SHARING FOR QUASI-STATIC LOADS: POST-STRENGTHENING TESTS

In the post-strengthening tests of Bridge No. 101 all displacements were measured relative to the ground and, thus, the effect of end support displacements is embedded in the results. Hence, the empirical load share calculation is not strictly applicable. For pre-strengthening load tests, load share was computed by using displacement data for each reference ("relative to the ground" and "relative to the displaced chord") for comparison [1,2,4]. The results for several cases examined showed that the difference in reference point had only a minor effect on the calculated load shares. Though not conclusive, the inference is the support displacements in Bridge No. 101 were "small." Cautiously, one might interpret that the end support displacements were small enough to not significantly distort the empirical load share results. Regardless, the data available from the post-strengthening tests of Bridge No. 1 are what they are and computation of load share based on them is the only computation possible.

The 2 mph results for Load Cases 1@2 mph - 4@2 mph of the post-strengthening tests were used to calculate load share based on the empirical method. Table 6 compiles the empirical stringer (termed "ply" in the table) load share results for each span for each loading case. Only Span 2 had all stringers instrumented in both chords. Thus, for that span, the load share values are tabulated for both chords. Only the East chord was monitored in Spans 1 and 3, so results are tabulated for that chord only. It is reiterated that the data for stringer 4 of Span 2 had suspect data for the 2 mph train passes. Table 6 tabulates the results if the suspect stringer 4 data are excluded. Without stringer 4 included in Span 2, the actual load share cannot be determined for the East chord. Instead "pseudo-load share values" are entered for plies 1, 2, 3 and 5. These were determined by ignoring stringer 4 and calculating the load share based on weighting the resistances of the remaining four stringers, relative to the total of their resistances. These values were then multiplied by 4/5 to get the tabulated entries. Stringer load shares ranged between 11 percent and 32 percent for the overall entries, including pseudo-values for the East chord, wherein the data for stringer 4 is suspect. If all stringers share load equally, they all would be at 20 percent.

In Table 6, plies 1 and 10 are the added stringers. It is noted that the added exterior stringers are actually four stringers, one two-span stringer and one single-span stringer per chord. So the degree of tightness of fit might vary between them. In the both chords, Span 1 had the added single-span including. Including the pseudo-load share values for the chord with the suspect stringer 4 data, the load share of the added stringers ranged between 16 percent and 29 percent, depending upon the load case.

TABLE 6. Load Sharing Proportions for Stringers (including Stringer 4 in Span 2) 1997 TEST @ 2MPH – LOAD CASES 1, 2, 3, & 4									
Bridge Span	**Ply**	**Load Sharing Proportions**							
		Load Case 1@2 mph		**Load Case 2@2 mph**		**Load Case 3@2 mph**		**Load Case 4@2 mph**	
		CF*MOE *D (10 ^{6} lb/in)	**Pro.**	**CF*MOE *D (10 ^{6} lb/in)**	**Pro.**	**CF*MOE *D (10 ^{6} lb/in)**	**Pro.**	**CF*MOE *D (10 ^{6} lb/in)**	**Pro.**
Span 1 (G-J) East Chord	**1**	-0.2987	**16%**	-0.1254	**18%**	-0.2822	**16%**	-0.1967	**16%**
	2	-0.4777	**25%**	-0.1600	**23%**	-0.4351	**24%**	-0.2780	**23%**
	3	-0.3140	**16%**	-0.1068	**15%**	-0.2956	**16%**	-0.1799	**15%**
	4	-0.4710	**24%**	-0.1809	**26%**	-0.4424	**25%**	-0.3116	**26%**
	5	-0.3624	**19%**	-0.1309	**19%**	-0.3464	**19%**	-0.2482	**20%**
Span 2 (D-G) East Chord	**1**	-0.0467	**17%**	-0.4556	**23%**	-0.2224	**24%**	-0.3559	**22%**
	2	-0.0627	**23%**	-0.5441	**27%**	-0.2419	**26%**	-0.4336	**27%**
	3	-0.0947	**35%**	-0.6227	**31%**	-0.2622	**28%**	-0.5384	**33%**
	4	-0.0358	**13%**	-0.0740	**4%**	-0.0777	**8%**	-0.0390	**2%**
	5	-0.0304	**11%**	-0.3074	**15%**	-0.1244	**13%**	-0.2429	**15%**
Span 2 (D-G) West Chord	**6**	-0.0474	**15%**	-0.3624	**19%**	-0.1826	**20%**	-0.3223	**19%**
	7	-0.0804	**25%**	-0.4889	**26%**	-0.2282	**25%**	-0.4652	**27%**
	8	-0.0836	**26%**	-0.3998	**21%**	-0.1788	**20%**	-0.3785	**22%**
	9	-0.0720	**22%**	-0.3090	**16%**	-0.1490	**17%**	-0.2707	**16%**
	10	-0.0374	**12%**	-0.3348	**18%**	-0.1585	**18%**	-0.2905	**17%**
Span 3 (A-D) East Chord	**1**	0.0211	**20%**	-0.0967	**29%**	-0.4635	**26%**	-0.0437	**28%**
	2	0.0117	**11%**	-0.0561	**17%**	-0.2775	**16%**	-0.0278	**18%**
	3	0.0297	**28%**	-0.0909	**27%**	-0.4496	**25%**	-0.0449	**29%**
	4	0.0233	**22%**	-0.0509	**15%**	-0.3103	**17%**	-0.0223	**14%**
	5	0.0188	**18%**	-0.0396	**12%**	-0.2879	**16%**	-0.0175	**11%**

Note: * = pseudo-load share values
= 4/5 (proportion based on 4stringers; string 4 excluded)
** = suspect data for stringer 4 is excluded

11. COMPARISON BETWEEN PRE- AND POST-STRENGTHENING RESULTS

A comparison of the load share results for the original and stiffened bridge is constrained by the limitation of the post-strengthening tests to ground-referenced displacements as well as the lack of extensive support displacement measurements. Given the above limitations, to attempt a rational comparison it is necessary to accept that the support motions were small enough to not affect the load shares. "Small enough" is intended as meaning, "If the use of ground-referenced displacement data yields similar load share findings to the pre-strengthening they may have been insignificant." As more stringers were added in the strengthening and the train configurations and axle loads were different for each test train, further assumptions are implied. In the pre-strengthening tests for Bridge No. 101 much less instrumentation was in place than in post-strengthening tests. In the latter case, one sequence of loadings data for one stringer was eliminated because of suspected transducer malfunction. Thus, many positions monitored in post-strengthening tests were either not included in or not usable from the monitoring in pre-strengthening tests. Thus, availability of similar loadings (pre-strengthening static vs. post-strengthening quasi-static) for which corresponding instrumentation locations existed for all stringers within a span-chord location was very limited. Only a few examples of specific load case to load case comparison were possible. Consequently, a general approach to comparing the calculated empirical load shares was utilized for the post-strengthening data.

Table 7 summarizes the pertinent empirical load shares, including the limited comparative results obtained from the pre-strengthening tests. Due to the constraints mentioned above, it is not possible to make a "one-to-one" comparison for individual load cases, so ranges are listed. For the two load tests and various loadings (static, TLV ramp, quasi-static) the range of load share values obtained are listed, together with an indication of the measurement reference for them. This tabulation constitutes the best available information for comparison of the load shares before and after addition of the helper stringers. The following paragraphs (taken, essentially, verbatim from the full report [3]) more directly examine if the ground reference used in the post-strengthening tests is a factor or not. To the extent possible, they also examine if the empirical load shares calculated for the post-strengthening results are reasonable.

For the East chord of Span 1, all tabulated values are based on displacements measured relative to the ground. Although the empirical method does not strictly apply, at least the data reference is the same. The pre-strengthening values for the TLV loading and the post-strengthening load shares for the 2 mph speed are consistent. Considering all stringers, as expected, the latter values (15 percent to 26 percent) are about 4/5 of the former values (21 percent to 34 percent). For the post-strengthening loads examined, the new stringers carried somewhat less load share then the old stringers. The pre-strengthening values for the West chord (20 percent to 35 percent) are essentially the same as for the East chord (21 percent to 34 percent).

For Span 2, the pre-strengthening values for the East chord for the static train loads (20 percent to 31 percent) and the TLV load (19 percent to 31 percent) are essentially the same. They are both based on data referenced to the displaced chord. For the West chord, the corresponding ranges of values (17 percent to 34 percent versus 19 percent to 32 percent) are close and the average values are the same. These load shares are also both based on measurements made relative to the displaced chord. The inference is the nature of the loading (static train vs. TLV ramp) did not affect the load shares.

TABLE 7. Summary of Load Share Results – Range of Values					
Span	**Chord**	**Stringers**	**1995 Static Loads**	**1995 TLV Loads**	**1997 2mph Loads**
Span 1 (G-J)	**East**	**All**		21 – 34% **g**	15 – 26% **g**
		Old		21 – 34% **g**	15 – 26% **g**
		New		N/A	16 – 18% **g**
	West	**All**		20 – 35% **g**	
		Old		20 – 35% **g**	
		New		N/A	
Span 2 (D-G)	**East**	**All**	20 – 31% **dc**	19 – 31% **g**	10 – 32% **g ***
		Old	20 – 31% **dc**	19 – 31% **g**	10 – 32% **g ***
		New	N/A	N/A	16 – 21% **g ***
	West	**All**	17 – 34% **dc**	19 – 32% **g**	12 – 27% **g**
		Old	17 – 34% **dc**	19 – 32% **g**	15 – 27% **g**
		New	N/A	N/A	12 – 18% **g**
Span 3 (A-D)	**East**	**All**		24 – 28% **g**	11 – 29% **g**
		Old		24 – 28% **g**	11 – 29% **g**
		New		N/A	20 – 29% **g**
	West	**All**		17 – 31% **g**	
		Old		17 – 31% **g**	
		New		N/A	

Note:

g = based on displacements measured relative to the ground
dc = based on displacements measured relative to the displaced chord
***** = pseudo-load share values
= 4/5 (proportion based on 4stringers; stringer 4 excluded)

For Span 2, the pre-strengthening TLV values (19 percent to 31 percent East chord, 19 percent to 32 percent West chord) referenced to the displaced chord for Span 2 are slightly less than the pre-strengthening values (21 percent to 34 percent East chord, 20 percent to 35 percent West chord) referenced to the ground for Span 1. Thus, in this case, there is only modest difference between these two results for the different measurement references. The inference is the support displacements may have been "small enough," but two different span configurations are involved.

For Span 2, the post-strengthening values for Span 2 are based on displacements referenced to the ground. The pseudo-load share values (10 percent to 32 percent) for the East chord are similar to the actual empirical values (12 percent to 27 percent) for the West chord. In the East Chord, the range is not close to being 4/5 of the pre-strengthening range. The lower (upper) extreme is about 1/2 (1/1) of the pre-strengthening value. For the West chord, the upper extreme is about 4/5 of the pre-strengthening value. The lower extreme is only about 3/5 of the pre-strengthening value. This suggests the pseudo load shares for the East chord may not be dependable. The caveat is that the data reference differs for each set of test data (pre- vs. post-strengthening).

For Span 3, both the pre-and post-strengthening data were referenced to the ground. The pre-strengthening load share ranges (24 percent to 28 percent East chord, 17 percent to 31 percent West chord) for the TLV load differed for each chord, but the average result was about the same for each. The post-strengthening load share ranges (11 percent to 29 percent) for the East chord had a much wider range than for the pre-strengthening values (24 percent to 28 percent), the latter being for the TLV load. The maximum observed was about the same (29 percent versus

28 percent). On average, the post-strengthening load share was about 6 percent lower than the pre-strengthening results. This is consistent with an expected 5 percent ideal difference (20 percent each for a five-stringer chord, 25 percent each for a four-stringer chord).

For Span 3, comparing the pre-strengthening values for Span 3 (24 percent to 28 percent East chord, 17 percent to 31 percent West chord) with the pre-strengthening values for Span 1 (21 percent to 34 percent East chord, 20 percent to 35 percent West chord), the pre-strengthening results had a much wider range of values for the East chord. For the West chord, the ranges were similar. Thus, it is unclear if both end-spans shared load similarly.

Some more collective, broader findings are evident, too. Based on displacement data measured relative to the ground in the pre-strengthening TLV tests, the empirically calculated load share for individual stingers in the four-stringer chords was between 17 percent and 35 percent. This is stated without regard to stringer continuity and location, as no identifiable pattern was observed. Thus, the ideal value of 25 percent per stringer was exceeded by as much as 10 percent.

Based on quasi-static loads and displacement data measured relative to the ground in the post-strengthening tests, the empirically calculated load shares for the five-stringer chords was 10 percent to 32 percent. This is stated without regard to stringer continuity and location, as no identifiable pattern was observed. Ignoring the pseudo-load shares calculated for some stringers, the range was 11 percent to 29 percent. These latter values are reasonably consistent with an expectation they would be about 4/5 of the four-stringer chord values observed in pre-strengthening (17 percent to 35 percent). Thus, the ideal value of 20 percent was exceeded by as much as 9 percent, ignoring the pseudo load share values.

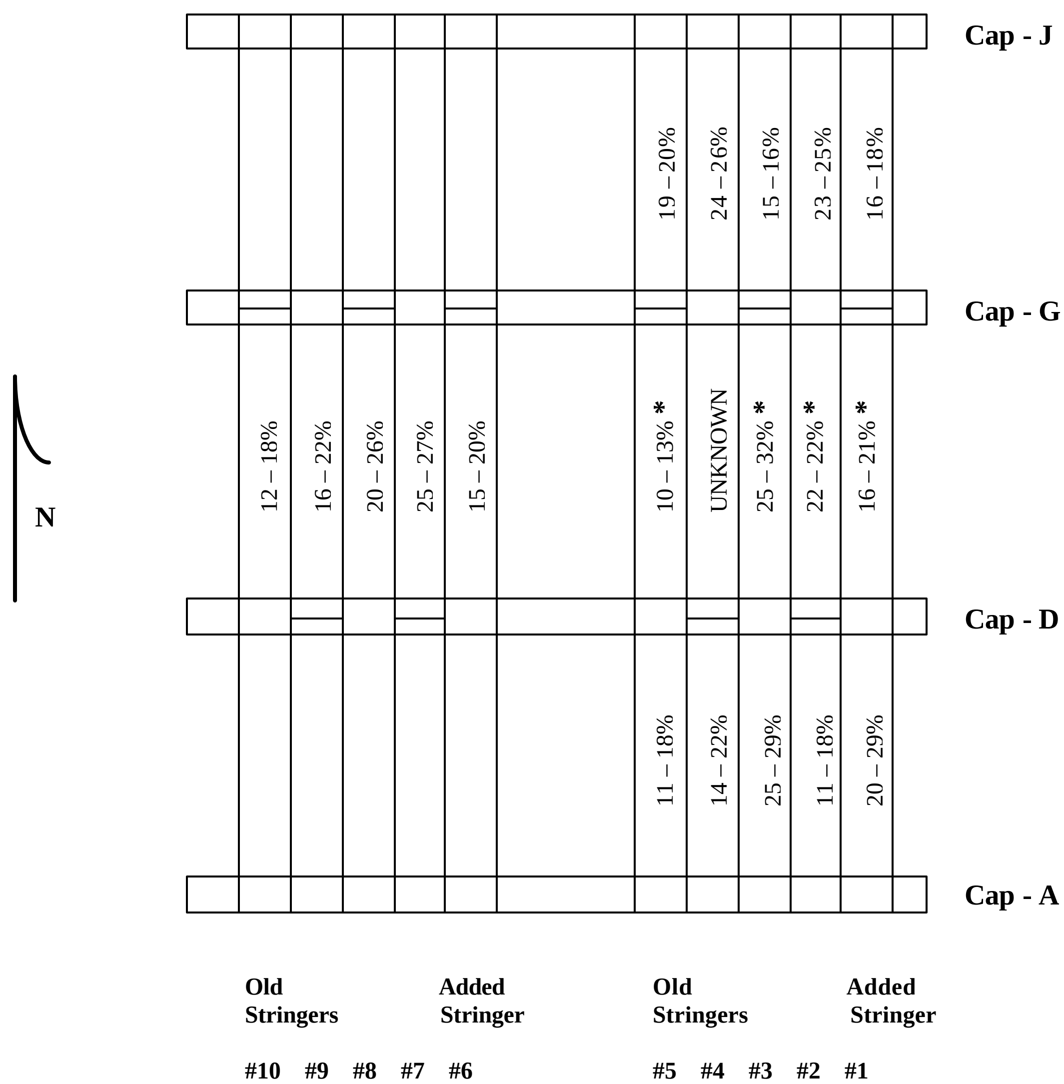
In the pre-strengthening tests, the load shares for ground referenced data in Spans 1 and 3 for the TLV loadings ranged between 17 percent and 35 percent for a four-stringer configuration. The load shares for the displaced chord referenced data for Span 2 for the TLV loadings ranged between 19 percent and 32 percent. The two are close in range and in average. The load shares for the pre-strengthening static load tests for displaced chord referenced data in Span 2 ranged between 17 percent and 34 percent. It is surmised that support motions may have been sufficiently small so as to not affect the empirical calculation of load shares.

Figure 20 is a schematic plan view of the bridge chords and stringers, with the post-strengthening stringer load share ranges indicated. All values are based on measurements referenced to the ground. Values for the East chord of Span 2 are the pseudo-load shares. Several points are evident. No pattern is evident for exterior stringers vs. interior stringers. For example, some middle stringers had the highest load shares but some did not. Some outermost stringers carried more load share than inner stringers. Predominantly, the single-span stringers carried less load share than the two-span continuous stringers. For Span 2, the magnitudes and pattern of the range of load share values is similar for both chords. The added stringers carried less load share than most of the other stringers in the respective chords. In part this may be due to being farthest away from the center of the loads i.e. from the rails. Another aspect that affects load share is the proximity of its ends to the piles of the end piers. Some bear directly over a pile, others do not (are either off center on a pile or in between two piles).

Based on the post-strengthening empirical results (from ground-referenced data), single-span stringers carry less load share (maximum of 22 percent, typically less than 20 percent) than two-span continuous stringers (max of 32 percent, typically greater than 20 percent).

Predominantly, load share taken by the added stringers compared to the old stringers (both in the strengthened bridge itself and with respect to the original bridge) appear to be moderately less than the ideal 20 percent load share.

Comparing the pre-strengthening load share data with that of the post-strengthening tests, it appears that support motions may not have been significant enough to make the measurement reference a factor in the above observations.



Note: * = pseudo-load share value = 4/5 (proportion based on 4 stringers; stringer 4 excluded)

FIGURE 20. Bridge No. 101 - Load Share Ranges Based on the Post-Strengthening Measurements

12. ROLLING TRAIN LOAD TEST RESULTS

The post-strengthening rolling train tests were conducted at higher speeds than the pre-strengthening tests, so the differences that resulted are of interest. Dynamic impact effects are examined by comparing the 20 mph and 2 mph responses. To allow a comparison, some of the detailed findings in the report on the pre-strengthening tests are extracted and included in the next section.

13. DYNAMIC RESPONSE

In pre-strengthening load tests, the bridge was subjected to the motion of the test train at two low speeds as pilot tests. Data were collected crudely by visual observation of train movement and timing by use of a stopwatch. Consequently, more rigorous and extensive rolling train testing was done in the post-strengthening load tests. Measurement of train position was done electronically, simultaneous with the acquisition of the displacement data. Extensive transient data was collected at all instrumentation locations for all train speeds and passes. As stated earlier, only the 2 mph and 20 mph data are relevant. The full report [3] examines this data for all spans of the bridge. In this abbreviated report, data for Span 2 are included as representative of the overall results.

13.1 Results

Figures 21a,b through 24a,b compare the responses at the 2 mph and 20 mph train speeds at mid-span of various stringers in Span 2 of the bridge. Figures 21a and 21b are for East chord stringers (stringers 1, 3 and 5) that are continuous into Span 3. Figures 22a and 22b are for the East chord stringers (stringers 2 and 4) that are continuous into Span 1. Figures 23a and 23b are for East chord stringers that are continuous into Span 3 (stringers 6, 8 and 10). Figures 24a and 24b are for the East chord stringers that are continuous (stringers 7 and 9). Predominantly, in each case, the amplitudes of corresponding peaks for each speed are essentially unchanged.

In Figure 21a (2 mph speed), the amplitude of the highest peak (peak 3) for stringer 1 (stringer 3, ply 5) is visually scaled as .315" (.325", .32"). The Figure 21b (20 mph speed), the corresponding amplitudes are visually scaled as .285" (.32", .285"). Using the ratios of these paired peaks, the dynamic impact effect is -9.5 percent (-1.5 percent, -10.8 percent). In Figure 21a, the amplitude of the lowest peak (peak 5) for stringer 1 (stringer 3, stringer 5) is visually scaled as .235" (.23", .23"). The Fig. 21b, the corresponding amplitudes are visually scaled as .225" (.225", .225"). The calculated dynamic impact effect is -4.2 percent (-2.2 percent, -2.2 percent).

In Figure 22a (2 mph speed), the amplitude of the highest peak (peak 3) for stringer 2 (stringer 4) is visually scaled as .315" (.135). The Figure 22b (20 mph speed), the corresponding amplitudes are visually scaled as .305" (.115"). Using the ratios of paired peaks, the impact effect is -3.2 percent (-14.9 percent). In Figure 22a, the amplitude of the lowest peak (peak 5) for stringer 2 (stringer 4) is visually scaled as .225" (.075"). The Figure 22b, the corresponding amplitudes are visually scaled as .225" (.070"). The calculated impact dynamic impact effect is 0 percent (-7.1 percent). However, the stringer 4 responses are suspect for reasons stated earlier in this report.

In Figure 23a (2 mph speed), the amplitude of the highest peak (peak 3) for stringer 6 (stringer 8, stringer 10) is visually scaled as .285" (.230", .225"). The Figure 23b (20 mph speed), the corresponding amplitudes are visually scaled as .245" (.21", .21"). Using the ratios of these paired peaks, the dynamic impact effect is -14.0 percent (-8.7 percent, -6.7 percent). In Figure 23a, the amplitude of the lowest peak (peak 5) for stringer 6 (stringer 8, stringer 10) is visually scaled as .21" (.175", .17"). The Figure 23b, the corresponding amplitudes are visually scaled as .195" (.165", .165"). The calculated dynamic impact effect is -7.1 percent (-5.7 percent, -2.9 percent).

In Figure 24a (2 mph speed), the amplitude of the highest peak (peak 3) for stringer 7 (stringer 9) is visually scaled as .25" (.245). The Figure 24b (20 mph speed), the corresponding amplitudes are visually scaled as .240" (.215"). Using the ratios of paired peaks, the impact effect is -4.0 percent (-12.2 percent). In Figure 24a, the amplitude of the lowest peak (peak 5) for stringer 1 (stringer 10) is visually scaled as .19" (.18"). The

Figure 24b, the corresponding amplitudes are visually scaled as .18" (.17"). The calculated impact dynamic impact effect is -5.3 percent (-5.5 percent).

Because the rate of data acquisition is the same for the two train speeds, true relative peaks may not have been captured. This may have contributed the negative dynamic impact effects. However, for Span 2, no positive values of the impact effect were observed, so impact is deemed negligible. Although graphical results for Spans 1 and 3 are omitted, summary observations are pertinent. For Span 1, the dynamic impact effect was predominantly negligible. In a few instances it reached about 2 percent for the highest peaks and 3 percent for the lowest peaks. For Span 3, dynamic impact was also predominantly negligible. In some instances it reached about 6-10 percent for the highest peaks. For the lowest peaks it was negligible except for one observed value of about 9 percent. The impact effect values would be somewhat higher if the offset in peaks between 2 mph and 20 mph plots were taken into account, i.e. if each 20 mph peak were compared with the exact displacement for the same train position for the 2 mph speed.

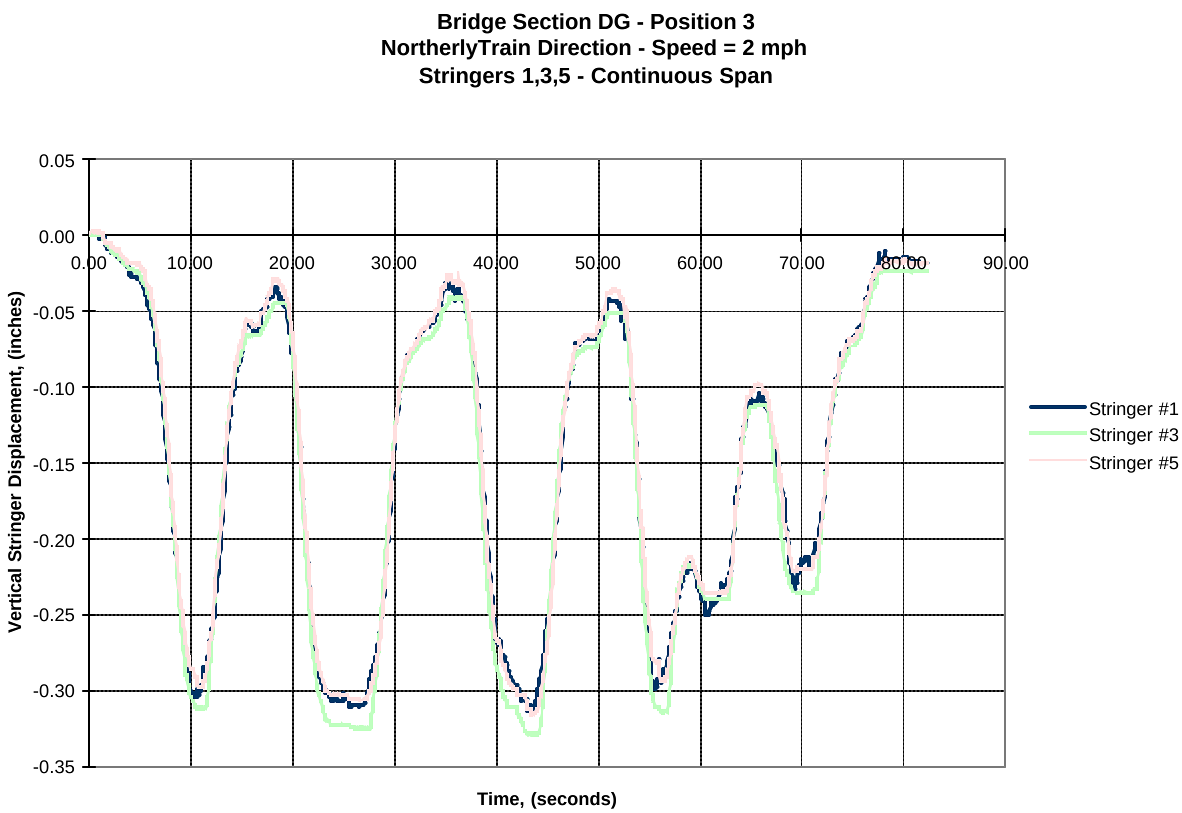


FIGURE 21-A. Transverse Response @ 2 mph

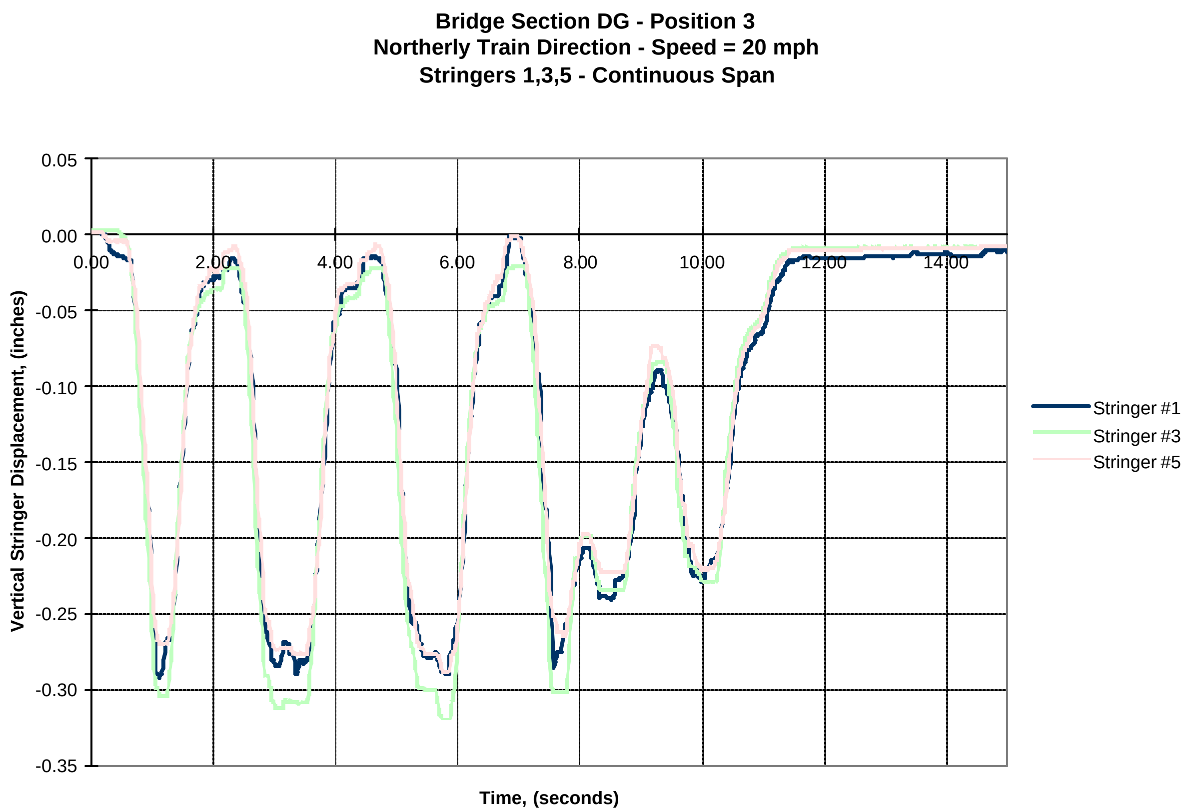


FIGURE 21-B. Transverse Response @ 20 mph

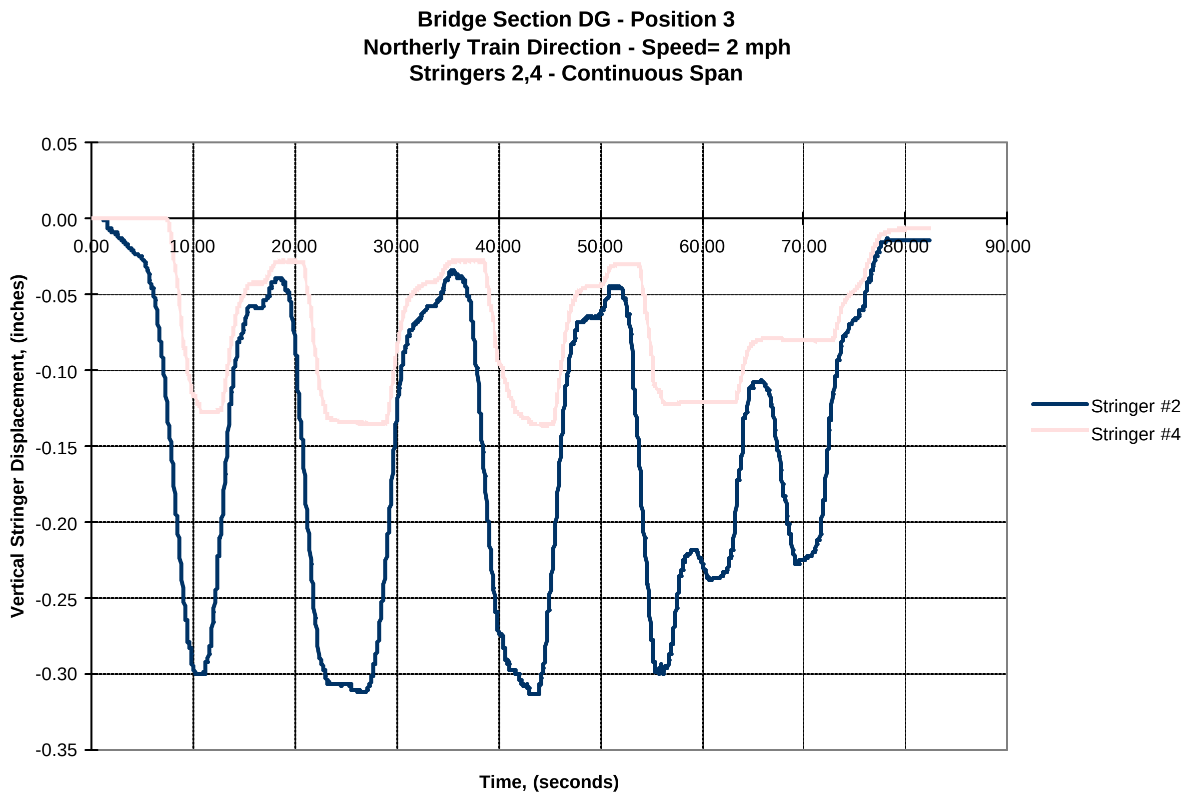


FIGURE 22-A. Transient Response @ 2 mph

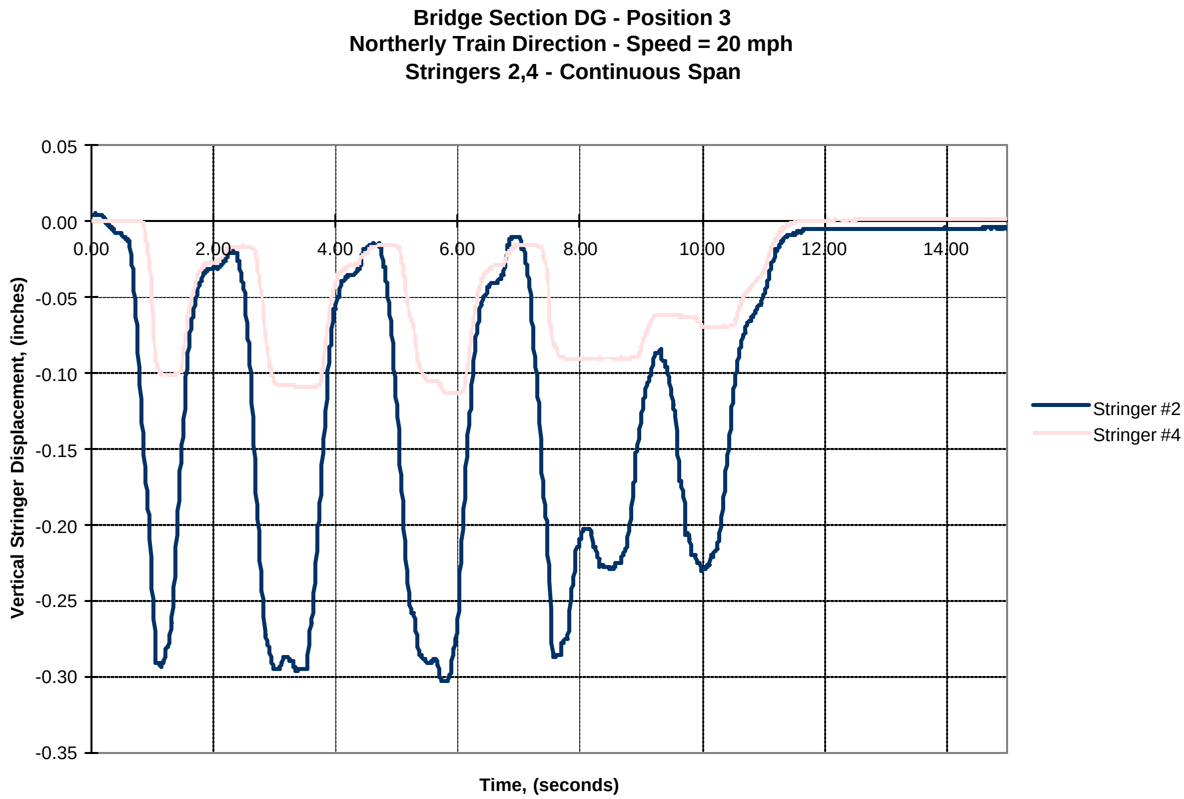


FIGURE 22-B. Transient Response @ 20 mph

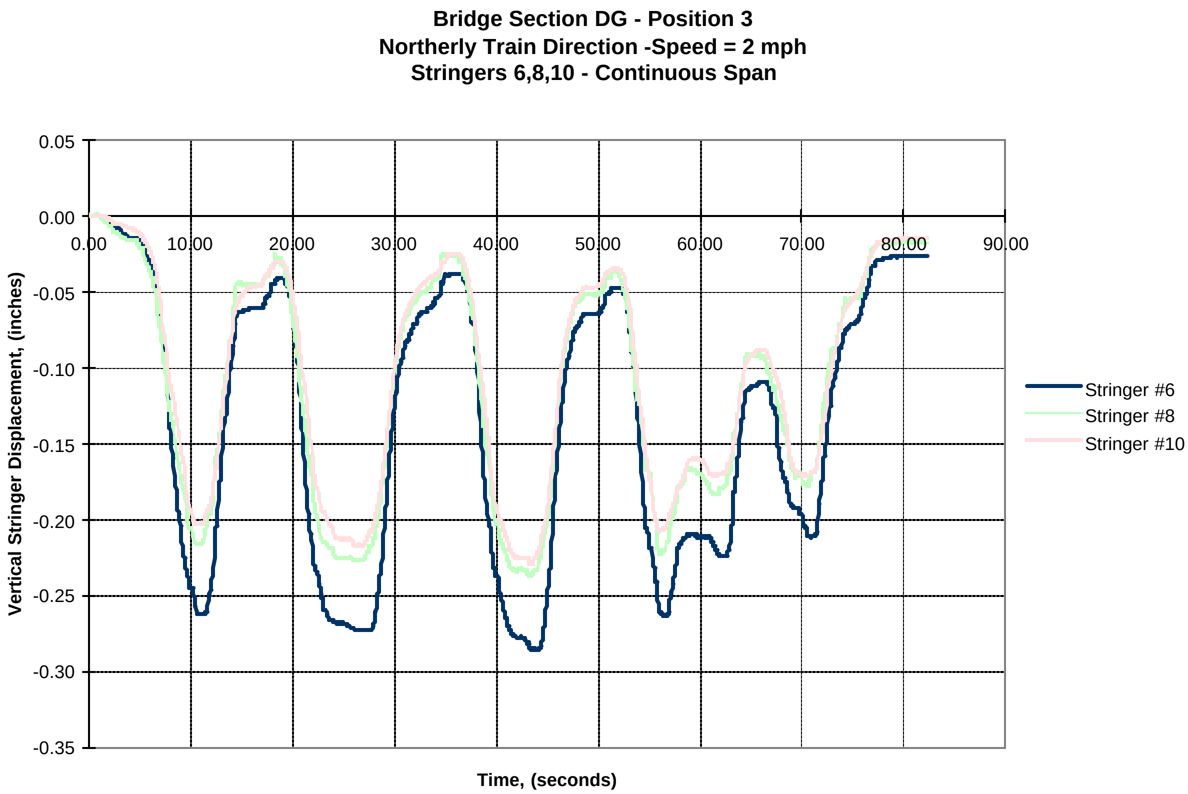


FIGURE 23-A. Transient Response @ 2 mph

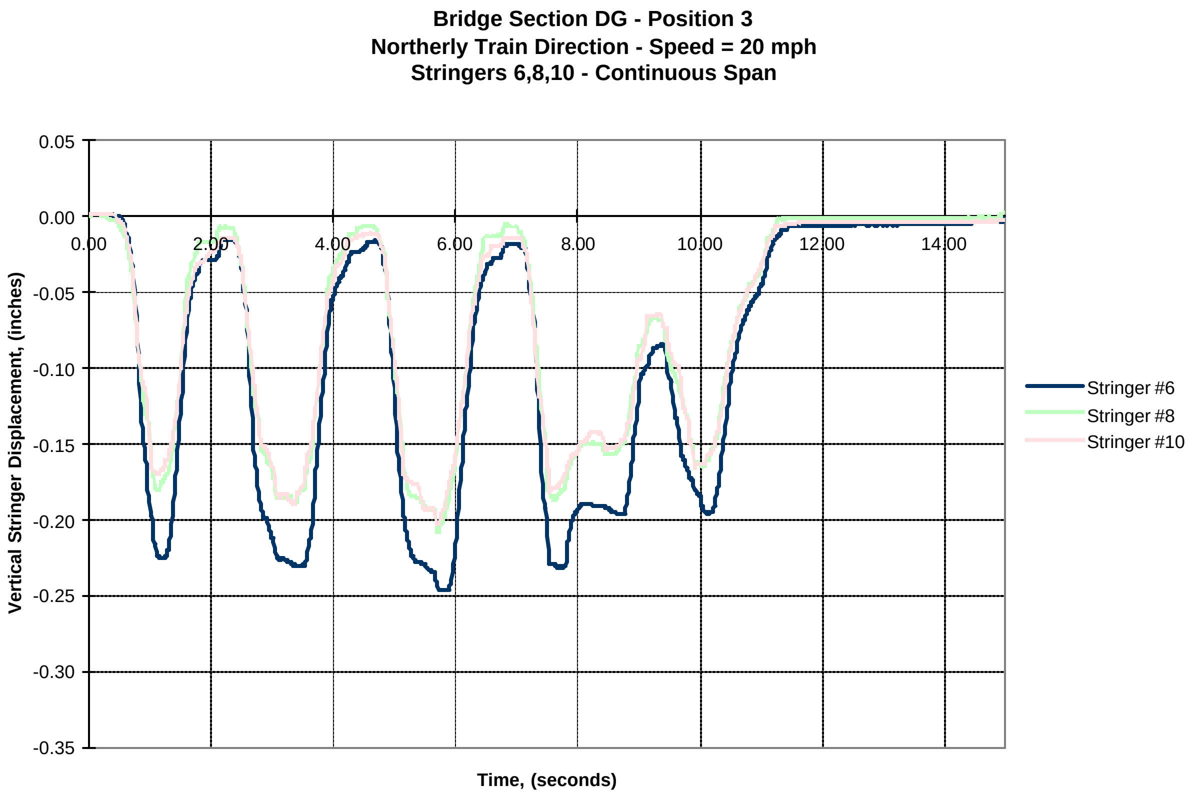


FIGURE 23-B. Transient Response @ 20 mph

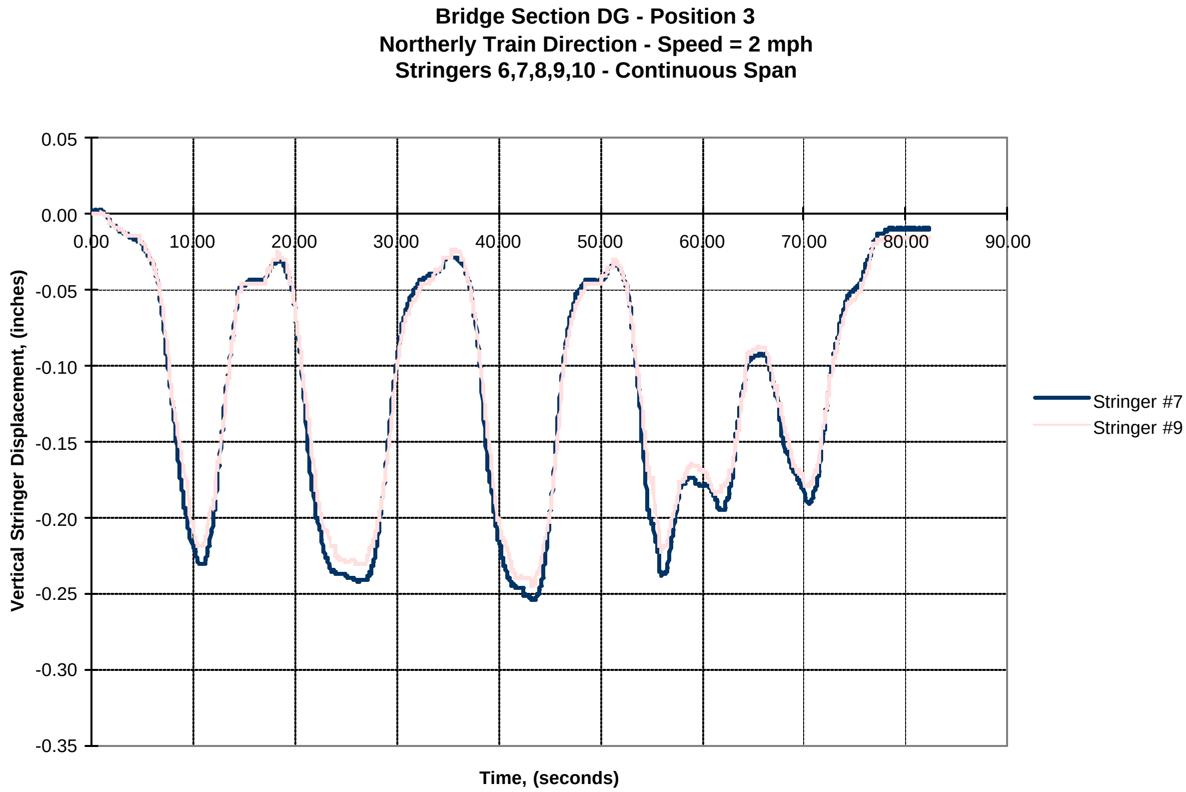


FIGURE 24-A. Transient Response @ 2 mph

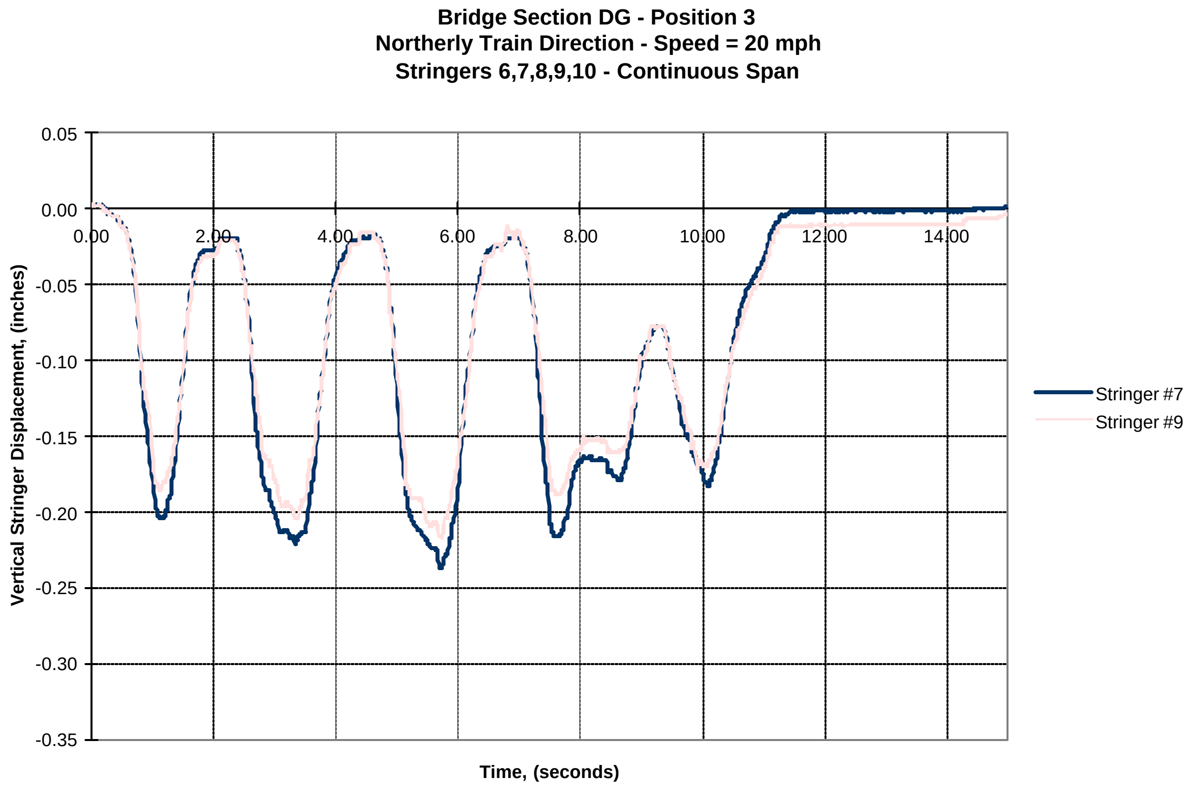


FIGURE 24-B. Transient Reponses @ 20 mph

14. OBSERVATIONS AND CONCLUSIONS

Comparison of the results of the pre- and post-strengthening test results leads to several general observations. Due to the use of electronic triggers to capture positions of the various axle, the dynamic impact effects observed in the post-strengthening rolling train tests are considerably more reliable than for the pre-strengthening tests. They showed dynamic impact to be either low or non-existent. No static trainload tests were done in the post-strengthening tests. Taking data for a few positions could have been readily done. Such data would be the correct base for examining dynamic impact effects. Nonetheless, the transient response at the extreme speeds (2 mph and 20 mph) and quasi-static load displacement results suggest that the 2 mph speed produced responses nearly equivalent to what static results would have produced. Data collected for the intermediate train speeds (5, 10 and 15 mp) are not included in this report. They are predominantly indistinguishable from that of the of the 2 mph and 20 mph extremes.

The finding of a high structural efficiency of the retrofit is not surprising. Considerable mechanical effort was required to install the added plies in between the cap and the ties, i.e. a tight fit was physically evident. It is not possible to distinguish between the effects of tightness of the retrofit stringers vs. the counter effect of not centering the ply chords under the steel rails.

Both CSU and the TTCI provided and independently installed potentiometers, control equipment and data acquisition capability and separately and jointly collected data. The fact that the interface of the two data basis produced consistent results lends increased credence to the post-strengthening test response data.

Primary conclusions from the test program are:

- The empirical procedure for calculating stringer load share produced rational results.
- An individual stringer in a four-stringer (five-stringer chord) chord can carry a maximum of 35 percent (29 percent) load share for the test loadings conducted in the two test programs.
- Based on pseudo-static loads and displacement data measured relative to the ground in the post-strengthening tests, the ideal load share value of 20 percent per stringer in the five-stringer chords was exceeded by as much as 9 percent in individual stringers, ignoring the pseudo-load share values for the East chord of Span 2.
- In the pre-strengthening TLV tests, the ideal value of 25 percent per stringer in the four-stringer chords was exceeded by as much as 10 percent in individual stringers. There was no pattern evident (e.g. interior stringers carrying more load share than exterior stringers or vice versa).
- There was no evident pattern of load share among exterior vs. interior stringers, but as expected, single-span stringers take less load share than two-span continuous stringers.
- The addition of exterior stringers stiffened the bridge chord about as expected. The additional stringers performed at an efficiency level between 82percent to 97 percent relative to the existing stringers.
- Dynamic transient responses in the post-strengthening tests for 2 mph and 20 mph speeds were predominantly the same.

- The post-strengthening test data show predominantly no dynamic impact effect, i.e., 6 percent to 10 percent increases occurred as isolated exceptions and some were for the lowest peaks. It was much less and often negligible for the preponderance of data. Thus, no concrete recommendation of a non-zero dynamic impact effect can be made for design code provisions. It appears to be very low to modest effect.

15. RECOMMENDATIONS

The following recommendations are made on the basis of this study.

- Regardless of position in the chord, a stringer in a four-stringer (five-stringer) chord was found to resist up to 35 percent (29 percent) of the chord loading. These values might be considered for design code adoption.

- Tests at speeds beyond 20 mph should be conducted to assess dynamic impact for service speeds above that level.

- Laboratory studies and/or detailed computer simulations are recommended to examine the separate consequences of not centering the helper stringers and of the degree of tightness of fit of the added stringers.

- Retesting the bridge via static positioning of the same test train as used in the pre-strengthening load tests would be useful.

- Load sharing is best examined by advanced analytical modeling, which allows for including the complexities of support motion, variability of wood member stiffness properties and relative movement between components (perhaps in a stochastic manner).

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