Test and Evaluation Report

Load Test to Assess Fatigue Detail Performance on Westbound I-80 Bridge over the Cedar River

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Prepared For

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BACKGROUND

On July 24, 2009 a load test was performed on the westbound I-80 Bridge over the Cedar River in Iowa. There are twin bridges at the location with dimensions of 1172 ft x 30 ft (see Figure 1). The westbound bridge was to be demolished soon after the load test for the construction of a new bridge. The bridge is multi-stringer (redundant) with welded plate girders (see Figures 2, 3 and 4). A fatigue crack was found in April 2009, which appears to have initiated at the top of the Pier 6 bearing stiffener on an exterior girder (see Figure 5). It is noteworthy that in 1981 a web fracture had been found on an exterior girder (at Pier 3) of the eastbound I-80 Bridge, which also appeared to have initiated from a fatigue crack at the top of the bearing stiffener (see Figure 6) but did not enter the top flange. At each of these locations, holes were drilled to arrest the crack growth. The locations of the 1981 fracture, the 2009 crack and the instrumentation and monitoring during the load testing are also shown in Figure 1.

The eastbound bridge was retrofitted as shown in Figure 7. It included drilling a ¾ in. hole on each side of the stiffener and removing the web material between the holes. Additional plates were bolted to the top flange and two vertical angles were bolted to the web and a bolster (bearing) was installed between the bottom flange and the pier cap in line with the angles.

OBJECTIVE

The primary objective of the load test was to determine the magnitude of strain in the web gap of the two exterior girders at Pier 3 of the westbound I-80 Bridge. In addition, the test was designed to determine potential “cause and effect” of the bridge deck interaction (including the associated “tipping” of the embedded top flange) with the web gap strain. An exterior girder and interior girder was also instrumented near the Pier 3 bearing area to determine the absolute magnitude of the negative region bending strain.

TEST SETUP

Pier 3

Prior to the placement of the instrumentation, feeler gages (minimum thickness was 0.008 in.) were used to assess the tightness of fit of the bearing stiffeners between the top of the stiffeners and the bottom of the top flange at all Pier 3 girders. The minimum thickness of the feeler gage could not be inserted at the top of stiffener/bottom of top flange interface at any of the Pier 3 bearing locations, and no visible evidence was noted of paint rust at the top of the stiffener region, indicating that the bearing condition was relatively tight (Figure 8). Subsequently, similar girder bearing stiffener locations were inspected, and feeler gages used, at Piers 4, 5 and 6. It should be noted that the girder bearing conditions at Piers 4 and 5 were similar to those at Pier 3 except for the following exceptions. At Pier 5 at an interior girder, the gap at the top of the stiffener/bottom of top flange interface was measured as 0.008 in. Paint rust at the top of the stiffener in the region was also visible at this location. Pier 6 is where the fatigue crack developed, and the fit at the top of the bearing stiffener is covered in a subsequent section later in this report.

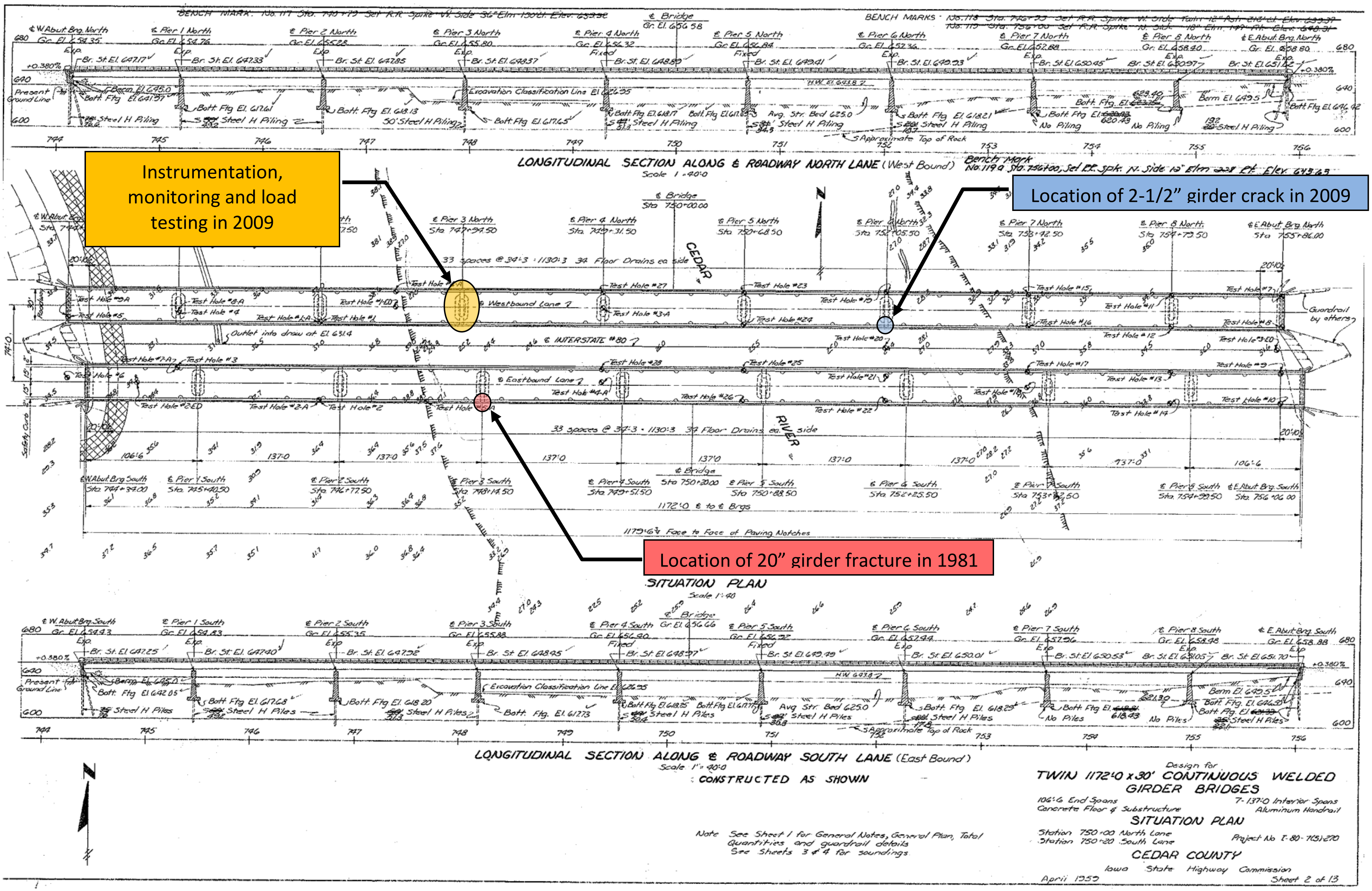


Figure 1. Situation Plan of twin bridges on the I-80 Bridge over the Cedar River.

Figure 2. Profile of one of the twin I-80 Bridges over the Cedar River.

Figure 3. View of underside of bridge. Note the channel diaphragms at the pier.

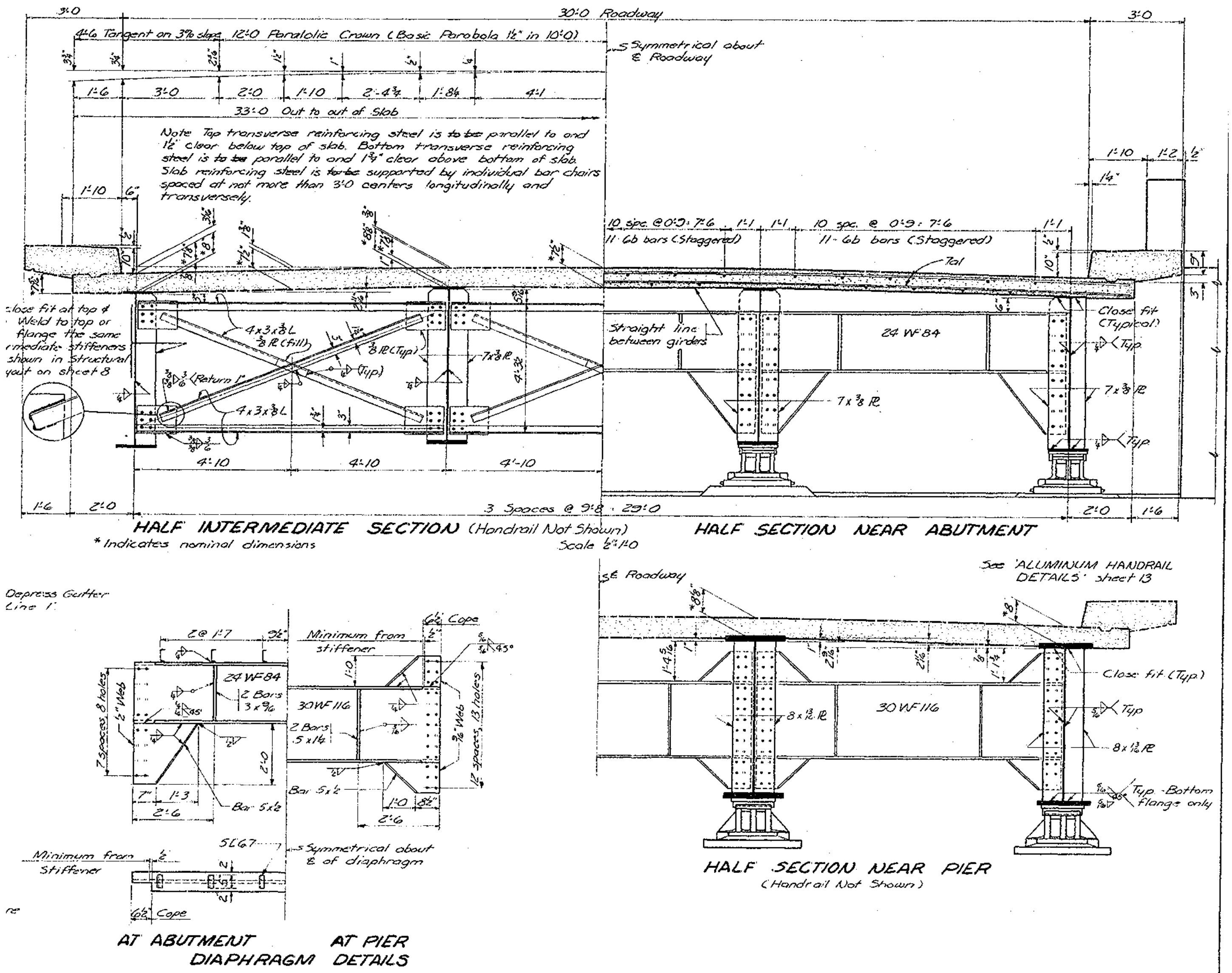


Figure 4. Bridge sectional views of I-80 Bridge over the Cedar River.

Figure 5. Fatigue crack on Girder 4 at Pier 6 of westbound I-80 Bridge over the Cedar River. The 2 ½ in. crack was found in April 2009.

Figure 6. Fracture of web on Girder 4 at Pier 3 of the eastbound I-80 Bridge over the Cedar River. In 1981, a 20" long fracture was found which was initiated at the top of the bearing stiffener at the pier

Figure 7. Retrofit repair of Girder 4 at Pier 3 of the eastbound bridge, including installation of a bolster (bearing) between the bottom flange and the pier cap in line with the angles.

Figure 8. Inspection and measurement (with feeler gage) of pier bearing stiffener fit with top flange of girder.

The strain and displacement instrumentation used at Pier 3 is shown in Figures 9 through 11. Gradient strain gages (consisting of 5 gages per tab and referred to as “Gradient” or “GS” in data plots) were placed in the web gaps near the top flange of the two exterior girders. At Girder 1 only the bottom 2 gradient gages (GS14 and GS15 at the bottom of the gradient tabs) were functional due to damage; all 5 gages were functional at Girder 4. Gage 1 (see Figure 10) was approximately 0.25 inches below the toe of the horizontal fillet weld; the distance between each of the four gages was 0.16 in., so the strain data given in the report will not reflect the maximum strain that would actually occur in the web gap near the toe of the weld. Vertical deck displacements measured relative to the top of the pier capbeam (referred to as “Displacement” or “Disp” in data plots) between Girder 1 and Girder 2, Girder 2 and Girder 3, and Girder 3 and Girder 4, were also monitored. Shown also in Figure 9 are LVDT “Tilt” measurements near the girder top flange edge on Girders 1, 2, 3 and 4. The tilt measurement utilized a displacement transducer oriented in the vertical plane to record flange tipping displacement relative to the bearing stiffener. Three girder strain gages (referred to in Figure 9 as “Strain”), one each at the top flange, bottom flange and mid-depth of the web, were placed on Girders 1 and 2 a distance d/2 from bearing support to quantify the flexural bending strain near the pier in the negative moment region.

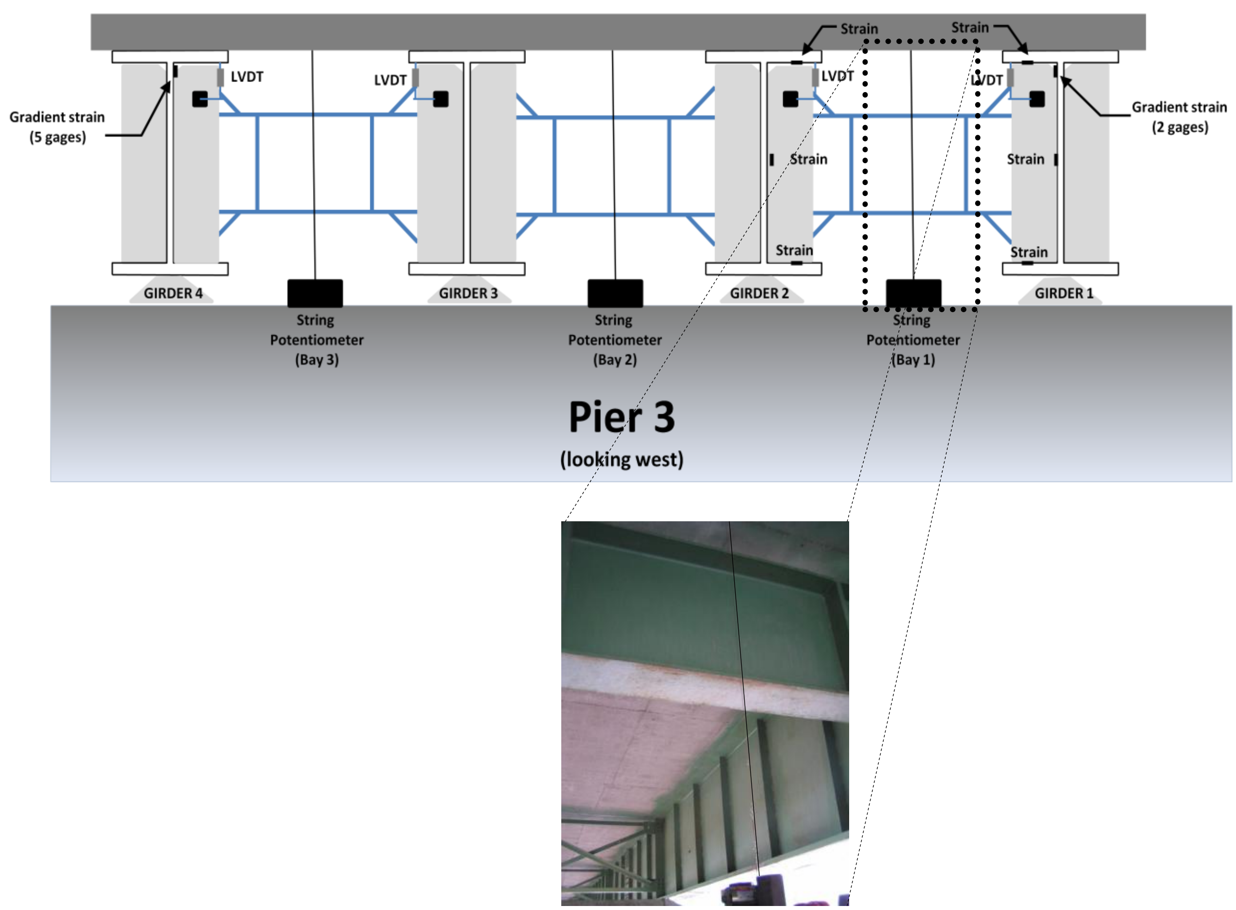


Figure 9. Schematic of the load test instrumentation at Pier 3 (looking west). Note that the three deck bay vertical displacements were made relative to the top of the pier cap beam.

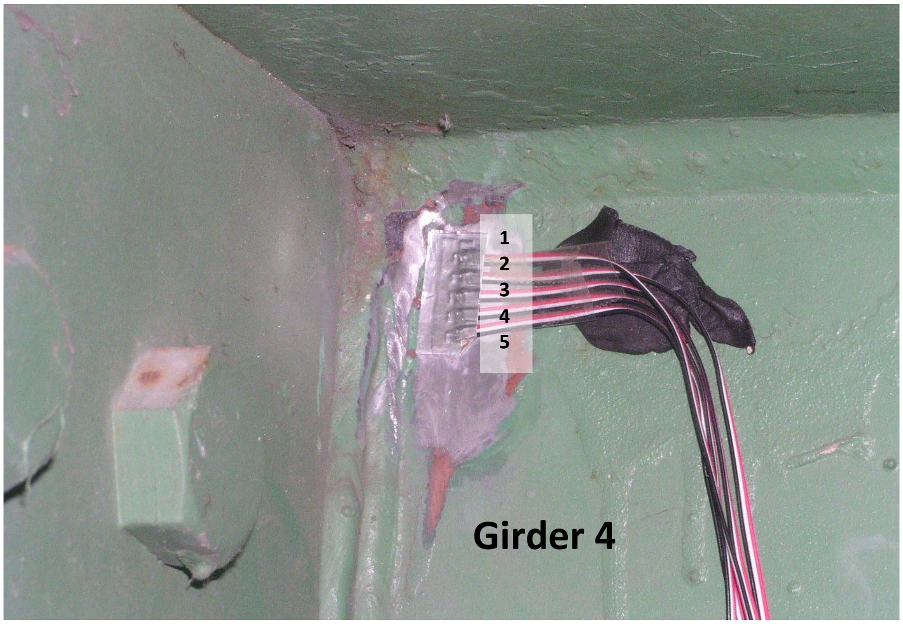


Figure 10. Typical gradient strain gage mounted on the inside faces of Girders 1 and 4 at Pier 3. Note the intersection of the vertical stiffener welds with the horizontal plate girder welds. The gradient was placed as close to the vertical and horizontal welds as possible. They are oriented to measure out of plane strain in the girder web.

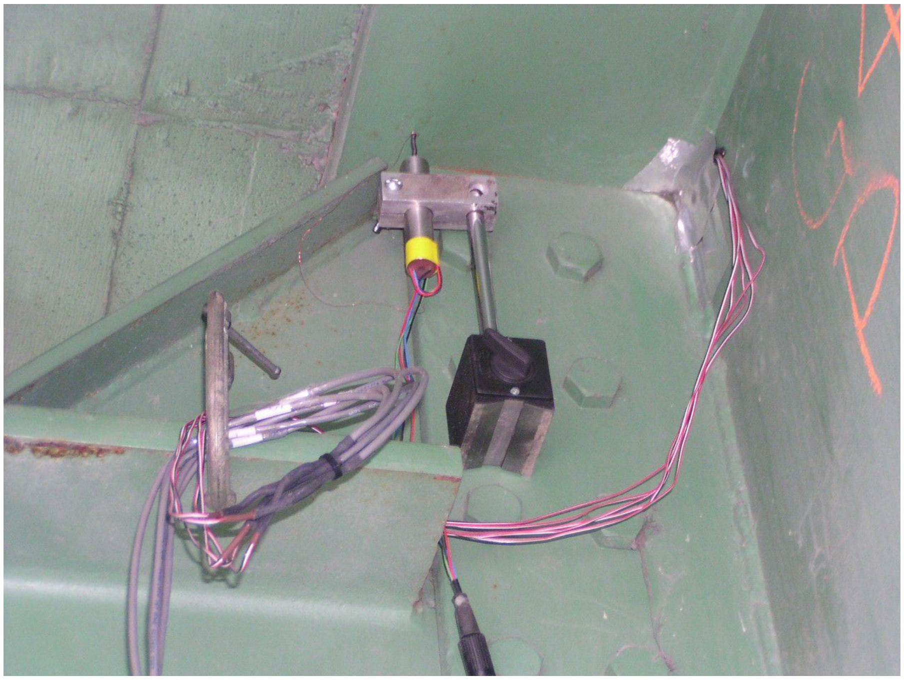


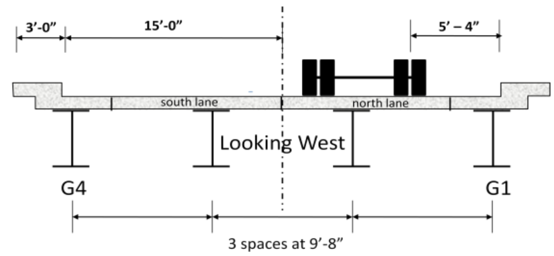
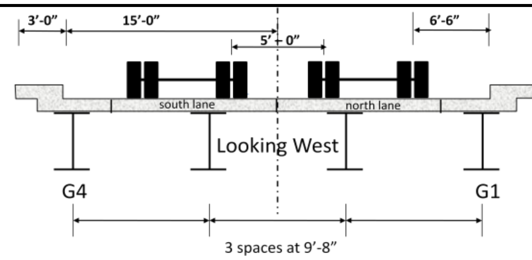
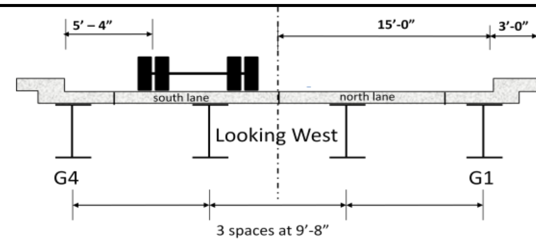
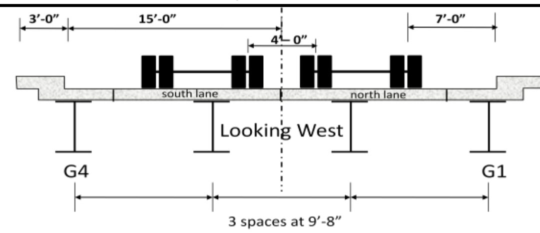
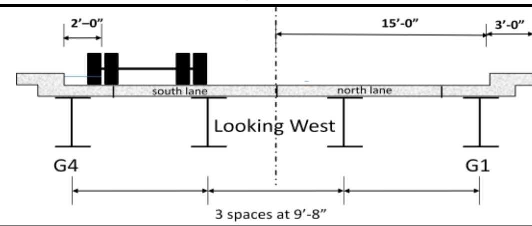
Figure 11. Displacement sensor (same location as Figure 10) to measure girder top flange tipping (vertical displacement also referred to as tilt in this report) relative to bearing stiffener. Similar set-up was used at all four girders at Pier 3.

The strain and displacement data were collected using a data acquisition system which used various sample rates of 20 to 60 Hz. The test load was controlled with the use of two standard loaded dump trucks from the Iowa DOT (Truck 1 and Truck 2, which had gross weights of 55.0 kips and 47.2 kips, respectively, Figure 12). A third truck was used (the Iowa DOT snooper seen in Figure 15, with a gross weight of 54.8 kips) along with the two dump trucks for several load cases only to collect data for the maximum girder flexural strain in the negative bending region near the pier. Otherwise, the two dump trucks were used in slow speed passes in various lane positions, including side by side (see Figure 13), back to back (see Figure 14) and single truck passes (see Table 1). Most truck passes were made with the trucks traveling in the middle of the marked lanes. Only one pass was made (Load Case 9) with truck ~2 ft from the face of the curb, and that was in the south lane.

Figure 12. Load test dump trucks; Truck 1 and Truck 2, respectively. Truck 1 had weights of 16.6 kips for front axle and 38.4 kips for rear tandem axles. Truck 2 had weights of 13.2 kips for front axle and 34.0 kips for rear tandem axles.

Figure 13. Load Case 8 looking east (trucks side-by-side in middle of lanes). Truck 1 was in south lane (right) and Truck 2 was in north lane (left).

Table 1. Load Case description for data collection at Pier 3. Truck(s) are traveling toward the west.

Load Case	Description	Visual Position
1	Snooper – Truck 2 – Truck 1 back-to-back-to-back about 20 feet apart traveling in the north lane	
2	Snooper – Truck 2 – Truck 1 back-to-back-to back about 10 feet apart traveling in the north lane	
3	Truck 1 in the north lane	
4	Truck 1 and Truck 2 back-to-back about 10 feet apart in the north lane	
5	Truck 1 in north lane, Truck 2 in south lane side-by-side with about 5 feet between them	
6	Truck 1 in south lane	
7	Truck 1 and Truck 2 back-to-back about 10 feet apart in the south lane	
8	Truck 1 in south lane, Truck 2 in north lane side-by-side with about 4 feet between them	
9	Truck 1 in south lane 2 feet from the curb	

NOTE: Load Case 9 was also used for data collection at Pier 6
NOTE: The center to center distance between the dual tires on the rear tandem axles is 6 ft

Figure 14. Load Case 7 (Truck 1 and Truck 2 back-to-back in middle lane). Trucks are separated by approximately 10 ft and are in the south lane.

Figure 15. Load Case 1 (Iowa DOT snooper, Truck 2 and Truck 1 back-to-back-to-back in middle lane). Trucks are separated by approximately 20 ft and are in the north lane.

RESULTS
Strain and displacements data plots are shown for selected critical data.

Figure 16 shows plots of gradient strain in the web gap on Girder 1, deck displacements in two "girder bays" and top flange tipping displacements on Girders 1 and 2. The loading consisted of the Iowa DOT snooper, Truck 2 and Truck 1 (Load Case 1) crossing the bridge approximately 20 ft apart in the north lane.

The maximum recorded strain was approximately 15 micro strain (equivalent to 0.45 ksi). The stress range (i.e. change from positive to negative stress) for the complete time record would only be slightly larger as the positive strain values are less than 2 or 3 micro strain. The GS14 and GS15 gradient gages (Figure 16a) are the two farthest down on the web away from the web/flange fillet weld.

The qualitative displacements of the deck in girder Bays 1 and 2 (negative in Bay 1 and positive in Bay 2) are reasonable given the test truck transverse wheel positions. The top flange tipping displacement also seems reasonable given the test truck transverse positions, and in a qualitative sense correlate well with the gradient strain and deck displacement records.

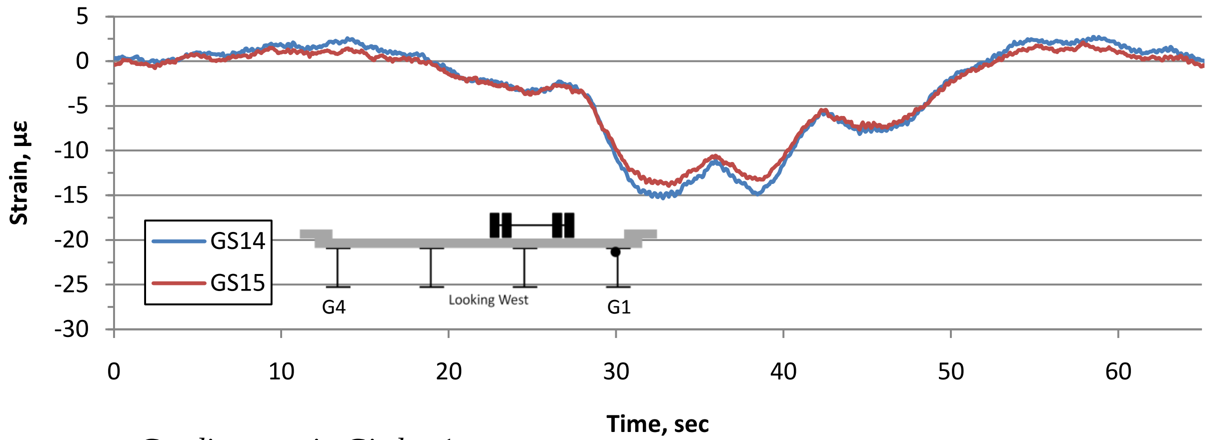
Figure 17 shows plots of gradient strain in the web gap on Girder 1, deck displacements in all three "girder bays" and top flange tipping displacements on Girders 1 and 2. The loading consisted of only Truck 1 (Load Case 3) crossing the bridge in the north lane.

The maximum recorded strain was approximately 10 micro strain (equivalent to 0.3 ksi). The stress range (i.e. change from positive to negative stress) for the complete time record would only be slightly larger as the positive strain values are less than 2 or 3 micro strain.

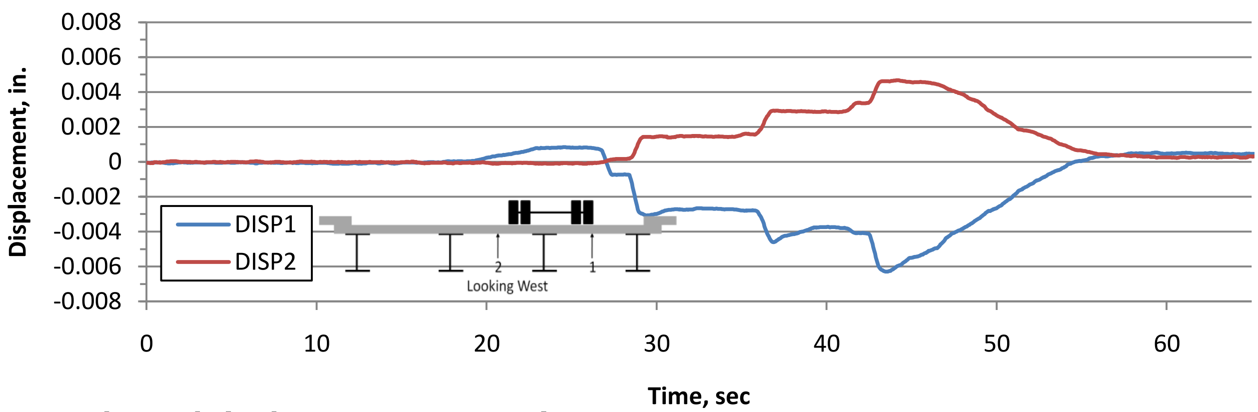
The deck displacement in the girder bays 1 and 2 seem reasonable given the test truck transverse wheel positions. The top flange tipping displacements also seem reasonable given the test truck transverse positions. Qualitatively the top flange tipping and deck displacement correlate well. However, in a qualitative sense the correlation between the gradient strain and both the deck displacement and the flange tipping displacement are not as clear as in Figure 16.

Figure 18 shows plots of gradient strain in the web gap on Girders 1 and 4, and the associated deck displacements in all three "girder bays". The loading consisted of Truck 1 in the south lane and Truck 2 in the north lane (side by side).

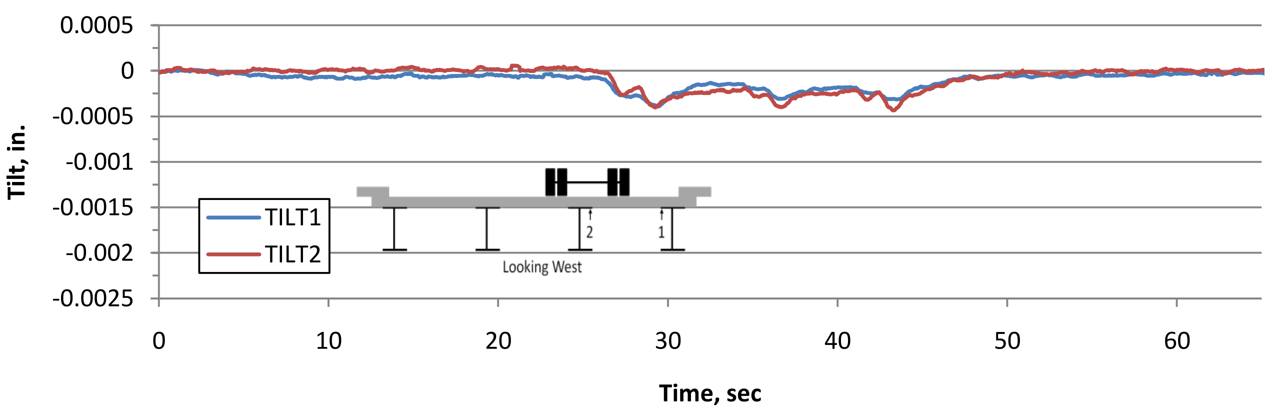
The maximum recorded strain in the web gap of Girders 1 and 4, respectively, was approximately 10 micro strain (equivalent to 0.3 ksi) and 28 micro strain (equivalent to 0.8 ksi). Note that at Girder 1, the two gradient strains should be correlated with GS 44 and GS 45 at Girder 4 since they are at similar positions in the web gaps. The stress range (i.e. change from positive to negative stress) for the complete time record would only be slightly larger as the positive strain values are less than 2 or 3 micro strain. It is interesting to note that the shape of the strain/time plot for web gap strain in Girder 1 and 4 are quite different, and certainly the maximum strain magnitudes differ as well. The displacements in girder Bays 1, 2 and 3 (negative in Bays 1 and 3 and positive in Bay 2) seem reasonable given the test truck transverse wheel positions.



a. Gradient strain Girder 1

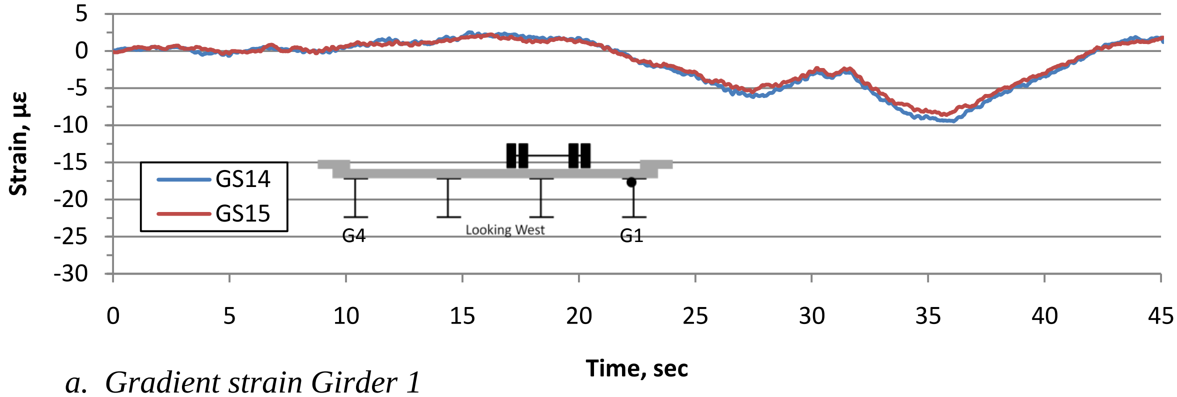


b. Deck displacements Bays 1 and 2

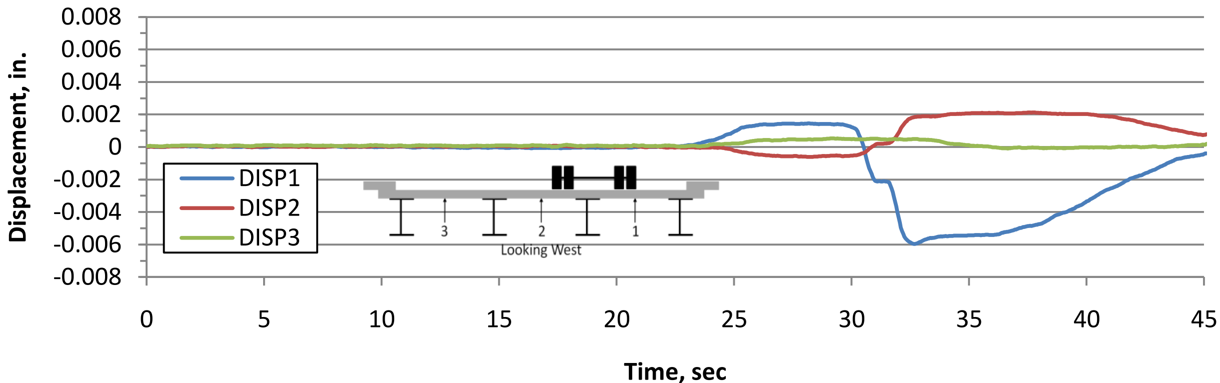


c. Flange tipping displacements Girders 1, 2

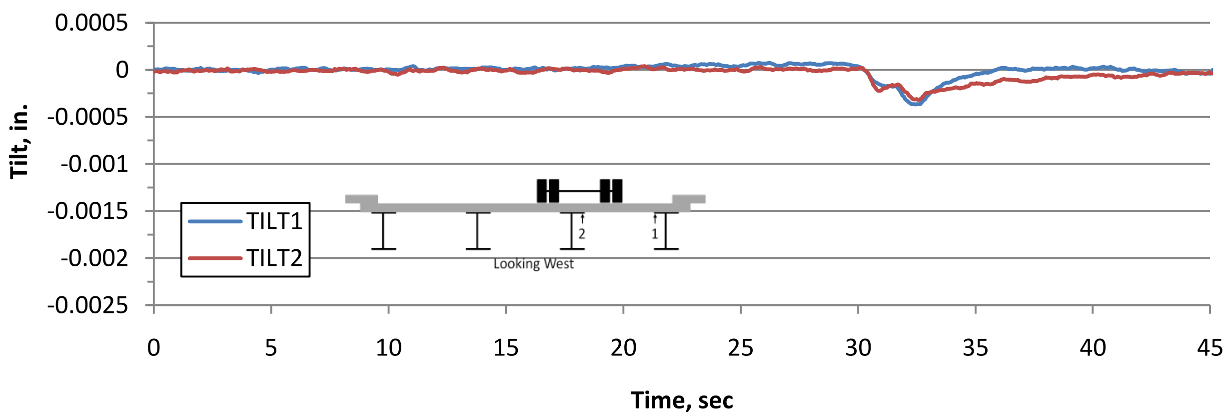
Figure 16. Load Case 1: Iowa DOT Snooper, Truck 2 and Truck 1 traveling back-to-back and separated by approximately 20 feet and traveling in the middle of the north lane.



a. Gradient strain Girder 1

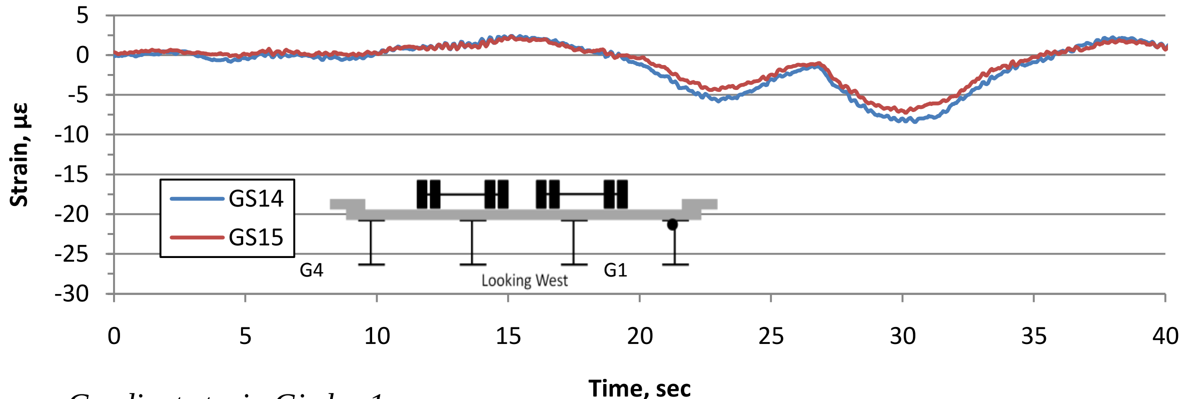


b. Deck displacements Bays 1, 2, 3

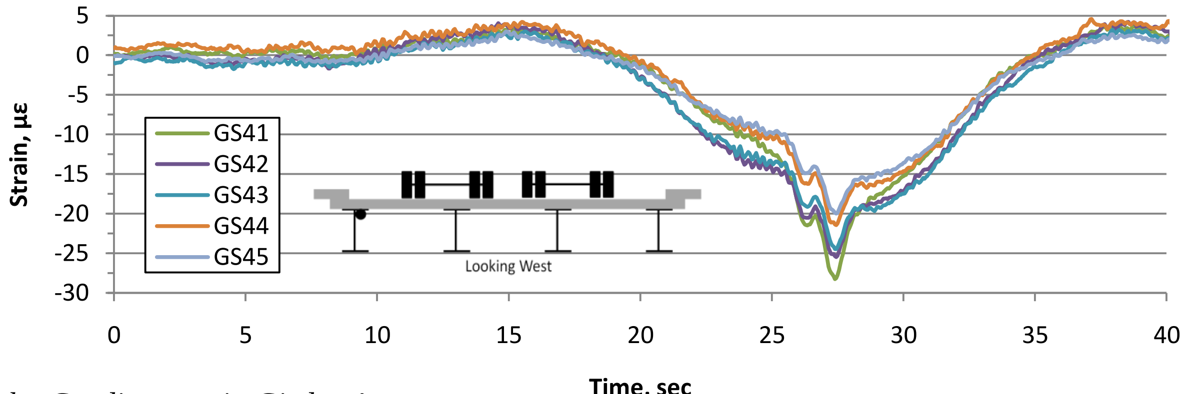


c. Flange tipping displacements Girders 1, 2

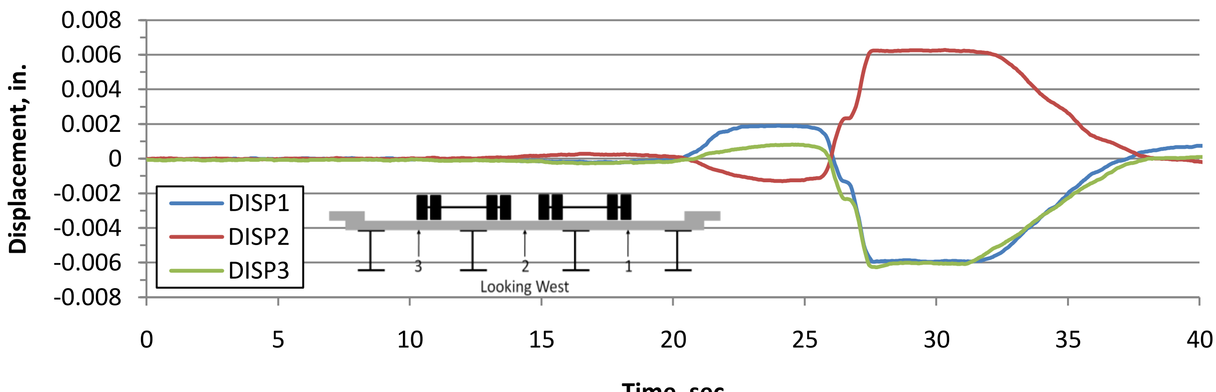
Figure 17. Load Case 3: Truck 1 traveling in the middle of the north lane (which would be approximately 5 ft-4 in. from the face of curb).



a. Gradient strain Girder 1



b. Gradient strain Girder 4



c. Deck displacements Bays 1, 2, 3

Figure 18. Load Case 8: Truck 1 in south lane, Truck 2 in north lane, side-by-side in middle of lanes.

General Observations at Pier 3

Based upon the results shown above for Pier 3, the following overall observations may be made.

- There is general correlation between the web gap strain and the top flange tipping (tilt), although the correlation seems clearer at Girder 4.
- There is general correlation between the web gap strain, top flange tipping and the bridge deck displacements, although the correlation seems clearer at Girder 4.

The broad inference of the observations above is that top flange tipping at the pier is caused at least partially by transverse bridge deck deformation. Subsequently, the top flange tipping induces web gap strains. However, for the data at Pier 3, the magnitudes of the strain range in the exterior girders are not relatively large (notwithstanding the fact that the measured strains would have been larger if the gradient gages had been placed nearer to the toe of the weld at the top flange). The maximum measured strain range corresponded to a stress range of 1.5 ksi (in Girder 4); the plot for this load case (Load Case 9; see Figure 19) was based on Truck 1 positioned in the south lane 2 ft from the face of the curb).

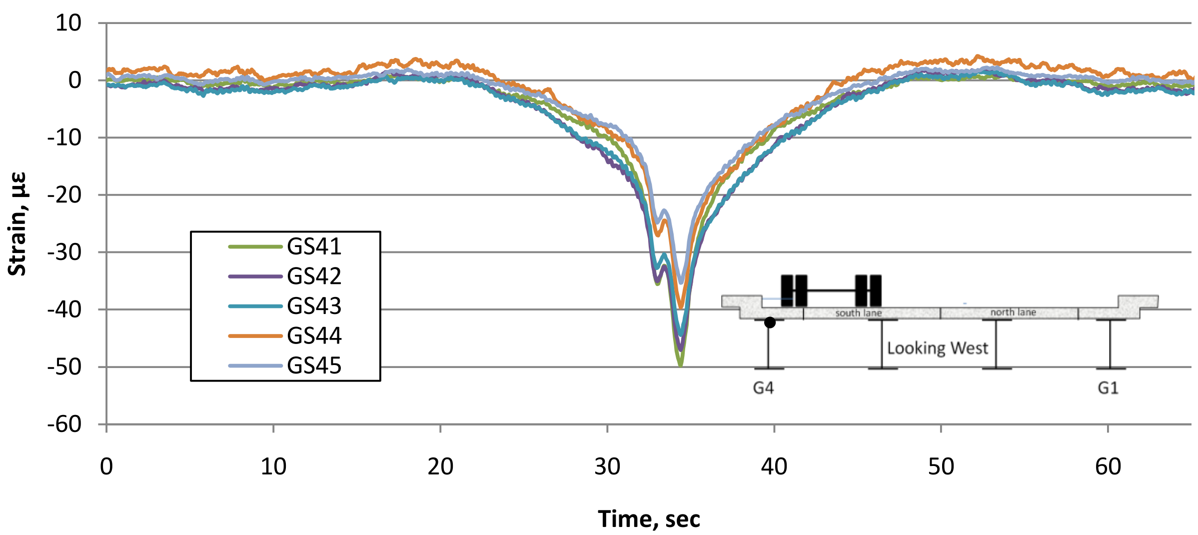


Figure 19. Girder 4 gradient strains for Load Case 9: Truck 1 in south lane positioned 2 ft from the curb.

ADDITIONAL DATA

Pier 6 Top Flange Tipping

Top Flange tipping (tilt) data only were collected at Girder 4 at Pier 6 on the westbound bridge. This is the location where the fatigue crack had occurred previously and where the drilled hole retrofit had been implemented (see Figure 5).

The same displacement transducer arrangement (described earlier in this report) at Pier 3 was used to measure girder top flange tipping at Girder 4 of Pier 6. Similar loading was applied on the bridge as for the data collection process at Pier 3. Prior to the placement of the instrumentation, feeler gages were used to assess the tightness of bearing stiffener fit at the girder top flange. At Girder 4, insertion of the feeler gage indicated a bearing stiffener fit that was not as tight as found at other piers (see Figure 20) on the bridge. The maximum measured gap was 0.014 in. Although the gap distance was minimally variable along the length of bearing, the feeler gage could be inserted all the way through the depth of the interface at every point along the length of bearing. Further, paint corrosion was visible at the stiffener/top flange interface.

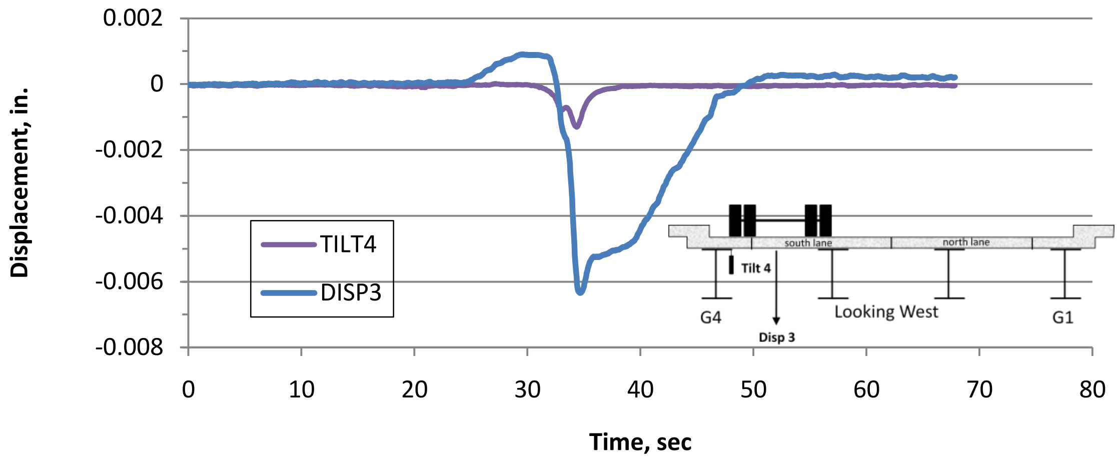
Figure 20. Feeler gage inserted between top of bearing stiffener and Girder 4 top flange at Pier 6. Note the corrosion between the interfaces and also note the retrofit hole that had been previously drilled to arrest a fatigue crack in the girder web.

Figure 21 shows plots of top flange tilt displacement on Girder 4 at Pier 3 and Pier 6 for the same loading. The loading consisted of Truck 1 in the south lane. Note that the Pier 6 girder flange tipping displacement was significantly larger than that for Pier 3.

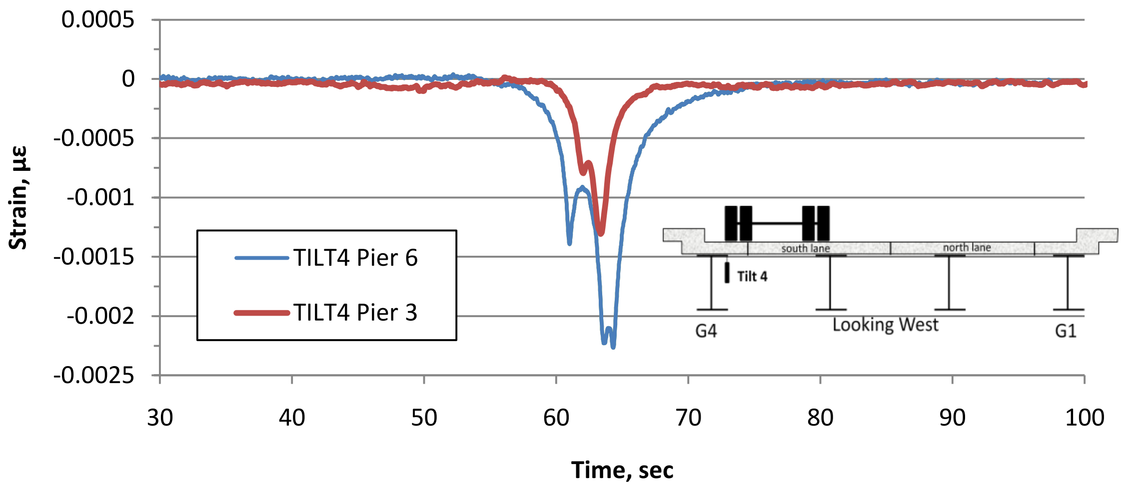
General Flange Tipping Observations at Pier 6

Based upon the results shown above for Pier 6, the following overall observations may be made.

- The bearing stiffener fit at the top flange of Girder 4 was not as tight as at other observed and measured (with feeler gages) locations on the bridge.
- The tipping of the top flange at Girder 4 at Pier 6 is larger than at Pier 3 for the same loading condition.



a. Flange tipping displacement for Girder 4 and deck displacement in Bay 3 at Pier 3



b. Flange tipping displacement for Girder 4 at Pier 3 and Pier 6

Figure 21. Truck 1 traveling in the south lane (which would be approximately 2 ft from the face of curb). See Load Case 9 in Table 1.

Pier 3 Negative Region Flexural Behavior

As shown in Figure 9 earlier in this report (and also below in Figure 22), and as described earlier in the instrumentation section of the report, Girders 1 and 2 were instrumented with 3 strain gages each (top flange, bottom flange and mid-depth of web) near Pier 3 to quantify the flexural behavior of the girders. Load Case 1 (Iowa DOT snooper, Truck 2 and Truck 1 traveling back-to-back-to-back in the middle of the north lane, and separated by approximately 20 ft) resulted in the largest compressive girder strain (bottom flange) of 58 micro-strain (approximate stress of 1.74 ksi) at a distance d/2 from the centerline of the pier bearing. Figure 23 shows for Load Case 1 at Girder 2 the strain profile. Figure 24 contains a strain history plot for the all three girder strain gages a distance of d/2 from the support, for Girder 1, Load Case 1. As noted, the largest tensile strain observed was 14 micro-strain (approximate stress of 0.42 ksi). Girder 1 also had the largest strain reversal from -56 to 14 micro-strain or a total strain range of 70 micro-strain (approximate stress of 2.1 ksi).

Figure 22. Strain gages B21, B22 and B23 on Girder 2 to assess flexural behavior near Pier 3.

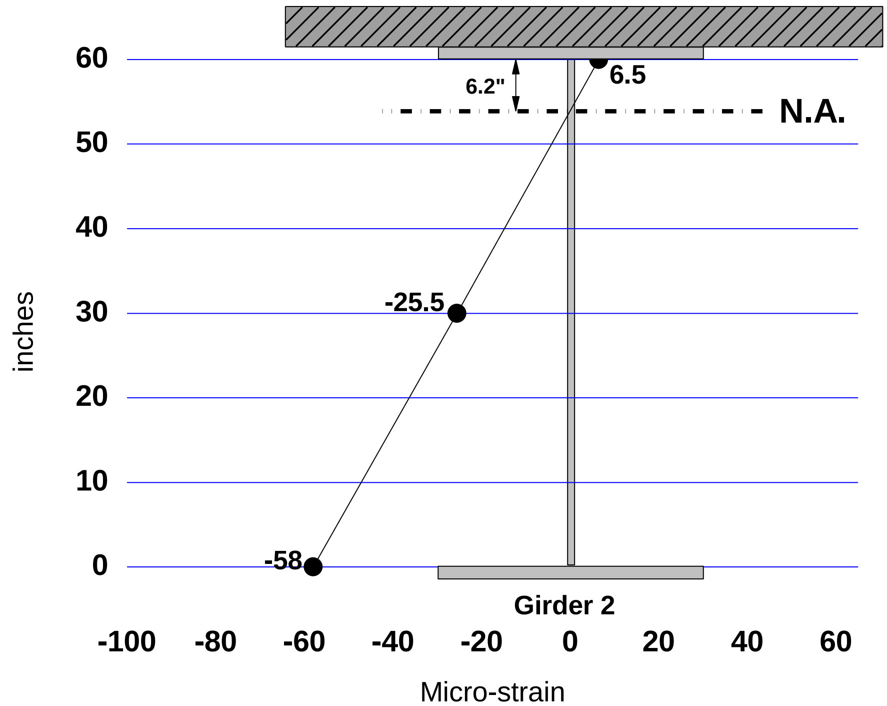


Figure 23. Neutral axis location for Girder 2 at Pier 3 determined from Load Case 1 data.

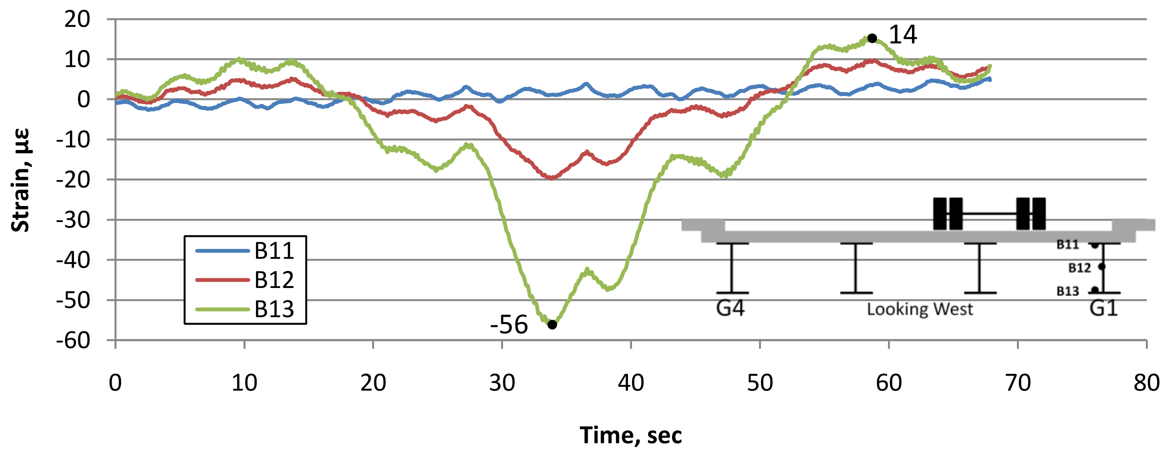


Figure 24. Girder 1 flexural strain for Load Case 1: Iowa DOT Snooper, Truck 2 and Truck 1 traveling back-to-back and separated by approximately 20 feet and traveling in the middle of the north lane.

CONCLUSIONS

Based upon the information presented above in this report, some global conclusions are:

- The top flange tipping of the girders at pier bearing locations generally correlate well with web gap strain and bridge deck displacement.
- The measured web gap fatigue stress range levels were relatively low (based on strain data).
- The tightness of the bearing stiffener tight fit at the girder top flange does appear to affect the fatigue stress level in the web gap, as noted when comparing flange tipping displacement data for Girder 4 at Pier 6 with Pier 3.

RECOMMENDATIONS

- Analytical evaluation to simulate the flange tipping at pier locations would be useful to better understand the influence of bearing stiffener fit-up on web gap strains.
- The implementation of a combined experimental and analytical study on the eastbound I-80 Bridge over the Cedar River prior to demolition could provide useful information. The analytical study could be initiated prior the experimental portion of that study to better develop an effective test plan. Complementary laboratory testing of girders removed from the bridge would also be very useful.