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Field Testing and Fatigue Evaluation of the Shippingport Bridge, Shippingport, PA

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FIELD TESTING AND FATIGUE EVALUATION OF THE SHIPPINGPORT BRIDGE
SHIPPINGPORT, PA

Final Report

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Ian C. Hodgson
Robert J. Connor
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Prepared for:
HDR, Inc.
Pittsburgh, PA

ATLSS Report No. 09-03

May 2009
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APPENDIX A – Instrumentation Plans
APPENDIX B – Development of Stress-range Histograms Used to Calculate Fatigue
             Damage
1.0 Introduction and Background

The Shippingport Bridge is located in Beaver County Pennsylvania and spans the Ohio River connecting the boroughs of Shippingport to the south and Midland to the north. The main river crossing consists of three-span combined deck/through truss, with span lengths of 341 feet, 620 feet, and 341 feet. The southern approach to the bridge consists of a two-girder two-span unit, while the northern approach is a single span multi-girder unit. The bridge was constructed in 1961 and has not gone through a major rehabilitation to date. A photograph of the bridge is shown in Figure 1.1.

Figure 1.1 – Elevation view of the Shippingport Bridge looking downstream (west)

Lehigh University’s ATLSS Center was contracted by the firm of HDR Inc. of Pittsburgh, PA to perform a detailed fatigue evaluation by measuring the in-situ behavior of the bridge. This portion of the work is part of a larger inspection/rehabilitation of this bridge under the direction of HDR. The instrumentation plan was developed in collaboration with engineers from HDR based on a review of the design and shop drawings, results of the in-depth inspection performed by HDR in August 2005, and computer analysis results produced by HDR. The locations selected for instrumentation are believed to be reasonably representative of all spans carried by the bridge. The scope of this work included controlled-load testing and long-term monitoring of the bridge. Estimation of remaining fatigue life at previously identified critical details was performed using the collected data. Retrofit strategies were to be developed where necessary.
2.0 Instrumentation Plan and Data Acquisition

The following section describes the sensors and instrumentation plan used during the controlled-load testing and long-term monitoring program. Detailed instrumentation plans can be found in Appendix A.

2.1 Strain Gages

Strain gages were placed at locations known to be fatigue sensitive and/or to provide insight into the global load distribution characteristics and general behavior of the bridge.

The majority of the strain gages installed in the field were produced by Measurements Group Inc. and were 0.25 inch gage length, model LWK-06-W250B-350. These gages are uniaxial weldable resistance-type strain gages. Weldable-type strain gages were selected due to the ease of installation in a variety of weather conditions. The “welds” are point or spot resistance welds about the size of a pin prick. The probe is powered by a battery and only touches the foil that the strain gage is mounted on by the manufacturer. This fuses the foil to the steel surface. It takes forty or more of these small “welds” to attach the gage to the steel surface. There are no arc strikes or heat affected zones that are discernible. There is no preheat or any other preparation involved other than the preparation of the local metal surface by grinding and then cleaning before the gage is attached to the component with the welding unit. There has never been an instance of adverse behavior associated with the use of weldable strain gages including their installation on extremely brittle material such as A615 Gr75 steel reinforcing bars.

These strain gages are also temperature compensated and perform very well when accurate strain measurements are required over long periods of time (months to years). The gage resistance is 350 ohms and an excitation voltage of 10 volts was used.

At web gaps in the south approach spans where very high strain gradients and limited space is available to install the gages, bondable strip gages were used. The strip gages were type EA-06-031MF-120 manufactured by Measurements Group Inc. Each strip gage contains five 120 ohm gages with a grid length of 0.031 inches. An excitation voltage of 5 volts was used. All gages were protected with a multi-layer weatherproofing system and then sealed with a silicon type compound.

2.2 Displacement Sensors

At several locations, relative displacement measurements were made between the deck and the truss in both the longitudinal and transverse directions. Linear Variable Differential Transformers (LVDTs) were used to record these displacements. These sensors were manufactured by Macro Sensors, Inc, model GHSD-750-250. These sensors are an all-welded stainless steel spring-loaded LVDT specially designed to be used in harsh industrial environments where dirt, water, and other contaminates may be present (such as a bridge). Hence, they are well suited for this application. The sensors have a stroke of ±0.25 inches. LVDTs of this type theoretically have infinite resolution, however, the resolution of the measurements was limited by the data acquisition system to 8x10^-6 inches as configured for this project.
2.3 Spans Selected for Instrumentation

Fatigue sensitive details were identified in both the main truss spans and approach spans. The instrumentation was concentrated in approach Spans 1 and 2, and truss Spans 3 and 4. It was determined that due to the large distance between the south approach spans (Spans 1 and 2) and the center span of the truss (Span 4), two data acquisition systems would be required. The first was located on the inspection walkway beneath the truss spans at Pier 3. All sensors installed in the truss spans were connected to this data acquisition system.

The second data acquisition system was installed on the inspection walkway at Pier 2 (at the south end of the truss spans). All sensors installed in Spans 1 and 2 were connected to this data acquisition system.

The location of all sensors was selected based on a review of the design and shop drawings, findings from the in-depth inspection performed by HDR, and a review of the results from an analysis and fatigue evaluation of the bridge also performed by HDR. The sensor locations chosen are believed to be representative of all similar details found throughout the bridge.

Extensive cracking has been well documented in the Truss spans. Termed Type “C” and “P” cracks, they are concentrated at the stringer diaphragms at the expansion joints and are the result of incompatible deformations between the deck and top of the floorbeams. Due to the presence of the diaphragm members (consisting of channel sections), these deformations are concentrated in the stringer web gap, resulting in high stresses. Type “P” cracks resemble a typical “smiley face” crack (typical of web gap cracking) at the base of the diaphragm. As a result of the drainage through the expansion joints, there is extensive corrosion of the stringer webs. This combined with the deformation described above results in the Type “C” cracks, which propagate horizontally just above the bottom flange juncture.

It had been originally proposed to install stain gages within the stringer web gap, however, due to the facts that (1) most existing connections had already cracked; (2) the phenomena which causes the cracking is well understood; and (3) these joints will likely be removed making the deck continuous, it was decided not to install these strain gages.

2.4 Summary of Sensor Locations

2.4.1 General

A total of 38 strain gages and eight LVDTs were installed. In the approach spans, 24 strain gages were installed. In the truss spans, 14 strain gages and eight LVDTs were installed. Complete instrumentation plans can be found in Appendix A.

Strain gages were installed at selected locations to measure the global response of the bridge. Additionally, a number of fatigue prone details previously identified were selected for instrumentation, and include the following:

Truss Spans

1. **Floorbeam/truss connection** – Two strain gages were installed on the top flange of a floorbeam adjacent to the full penetration weld to the endplate at the floorbeam-truss connection. Stresses resulting from in-plane and out-of-plane can be evaluated.

2. **Floorbeam at midspan** – Strain gages were installed at the top and bottom flange of Floorbeam 4 at midspan.
3. **Stringer/diaphragm connection** – A fatigue analysis performed by HDR indicated that this connection does not have sufficient remaining fatigue life. Two locations were instrumented to allow for a fatigue life calculation based on in-situ stresses.

4. **Longitudinal Stiffener Terminations** – Cracking has been found at the termination of a longitudinal stiffener at a web splice plate. Strain gages were installed back-to-back at two similar but uncracked locations.

5. **Floorbeam web cope** – The webs of floorbeams have a sharp-radius cope at the connection to the main girders. Fatigue cracking has been found at these locations (Type “T” cracks). Strain gages were installed back-to-back at two uncracked locations.

6. **Gusset Plate Termination** – Strain gages were installed back-to-back on the girder web at the termination of the gusset plate-to-web weld, a category E detail. Two locations were instrumented.

7. **Floorbeam knee brace** – Cracking has been observed at the floorbeam knee brace (Type “K” crack). A single strain gage oriented vertically was installed on two knee braces.

8. **Web gaps at floorbeam connection plates** – The floorbeam connection plates are not welded to the girder tension flange. A small web gap is formed at the connection plate snipe. Strip gages were installed at the web gap.

Figure 2.1 contains a photograph of the center span of truss looking north, away from Pier 3. The truss is a three-span continuous unit and is a combination of a deck and through truss. The floor system consist of floorbeams and stringers supporting a concrete deck. There are deck joints generally every fourth floorbeam. Shown in Figure 2.2 is a photograph of the floor system in Span 3 (the other spans are similar). As can be seen, there are six stringers. Note that there is a inspection walkway the complete length of the truss spans between stringers S4 and S5. The floorbeams frame into the truss verticals.
Figure 2.1 – View of center truss span (Span 4) looking north from Pier 3

Figure 2.2 – View of truss floor system in Span 3 looking south from Pier 3 (other spans similar)
The two southern approach spans (Spans 1 and 2) are comprised of a two-span continuous two-girder unit. A photograph of Spans 1 and 2 looking south from Pier 3 is shown in Figure 2.3. The floor system consists of floorbeams and stringers supporting a concrete deck. At the floorbeam-to-girder connection, the bottom flanges of the floorbeams are supported on knee-braces, as shown in Figure 2.4. These spans have a lateral bracing system which frame into a gusset plates welded to the girder webs (Figure 2.4).

Figure 2.3 – View of Spans 1 and 2 looking south from Pier 3
Figure 2.4 – Floorbeam-to-girder connection in Spans 1 and 2
2.4.2 Floorbeam-to-truss Connection

In the truss spans, the floorbeams frame into the truss verticals. The connection consists of a welded endplate that is bolted to the vertical. The weld between the floorbeam and the end plate is a complete joint penetration (CJP) weld. Two strain gages were installed transversely on the top flange of Floorbeam FB4 directly adjacent to the top flange-to-endplate weld (one gage on each side of the flange) as shown in Figure 2.5. These gages were installed to measure both the in-plane and out-of-plane bending stresses in the floorbeam. The stresses at this connection are needed to study the effects of the incompatibility between the deck and floor system. Furthermore, it was desired to perform a fatigue evaluation of the CJP weld. Installation of two gages allowed for the determination of both in-plane and out-of-plane effects so the governing loading mechanism driving the fatigue cracks can be identified.

![Figure 2.5 – Two strain gages installed at top flange of floorbeam at connection to truss vertical](image-url)
2.4.3 Longitudinal Stiffener Termination

Figure 2.6 contains a photograph of a typical installation of strain gages at the termination of a longitudinal stiffener in Span 1 at a bolted web splice. Cracking was found at a similar location during routine inspections. Two uncracked locations (one on each of the main girders) were instrumented. Note the strain gages were installed in a longitudinal orientation on both sides of the girder web, and butted against toe of the stiffener-to-web weld. These gages were installed to determine the governing loading mechanism which caused the observed cracking. The use of two gages (one on each side of the girder web) allows for the determination of both in-plane and out-of-plane stress components in the girder web at the termination of the stiffener, and therefore the critical loading mechanism driving the fatigue cracks can be identified.

![Figure 2.6 – Strain gage installed at the termination of a longitudinal stiffener at a bolted web splice (identical gage on other side of web).](image)

Figure 2.6 – Strain gage installed at the termination of a longitudinal stiffener at a bolted web splice (identical gage on other side of web).
2.4.4 Floorbeam Web Cope

The top flange and web of floorbeams in Spans 1 and 2 are sharply coped at the connection to main girders. Cracking has been found propagating out of the sharp cope radius in the floorbeam web. Strain gages were installed horizontally back-to-back on the web (i.e., on each side of the web) of floorbeams at the cope, as shown in Figure 2.7. Two floorbeam connections were instrumented: the upstream end of Floorbeam FB10, and the downstream end of Floorbeam FB9. Again, the use of two gages (one on each side of the floorbeam web) allows for the determination of both in-plane and out-of-plane stress components in the floorbeam web at the flange cope, and therefore the critical loading mechanism driving the fatigue cracks can be identified.

Figure 2.7 – Strain gage installed on coped web of floorbeam in Span 1 (identical gage on other side of girder web)
2.4.5 Gusset Plate Termination

The lateral bracing present in Spans 1 and 2 frames into the webs of the main girders at welded gusset plates located near the bottom flange. Strain gages were installed at the termination of the gusset plate at two locations in Span 2: at the downstream end of Floorbeams FB9 and FB10. At each location the strain gages were installed horizontally back-to-back adjacent to the gusset-to-web weld. Figure 2.8 contains a photograph of a typical installation. Again, the use of two gages (one on each side of the girder web) allows for the determination of both in-plane and out-of-plane stress components in the floorbeam web at the flange cope, and therefore the critical loading mechanism driving the fatigue cracks can be identified.

![Strain gage installed at the termination of the gusset plate](image)

Figure 2.8 – Strain gage installed at the termination of the gusset plate (identical gage on other side of girder web)
2.4.6 Floorbeam Knee Brace

Cracking was found at the knee brace-to-floorbeam weld during routine inspections. Strain gages were installed on the knee brace adjacent to this weld at two uncracked knee braces: at the upstream and downstream ends of Floorbeam FB10. Figure 2.9 shows a typical installation (note that the strain gage is oriented along the primary axis of the knee brace, perpendicular to the fatigue cracks at the weld toe). These gages was installed to study the stress field in the knee brace so the stress conditions driving the fatigue cracks can be understood.

Figure 2.9 – Strain gage installed on floorbeam knee brace in Span 2
2.4.7 Longitudinal LVDTs

Displacement sensors (LVDTs) were installed at four locations to measure the relative longitudinal displacement between the truss and the deck. As described previously, there are joints in the deck system generally every fourth floorbeam. The purpose of this instrumentation is to study the behavior of the bridge as-designed (with deck joints) and to utilize the data to calibrate a computer model of the bridge (performed by HDR) to study the effect of making the deck continuous for the full length of the bridge.

Two four-span deck units were selected for investigation: one in a back truss span (PP4 to PP8 in Span 3) and one in the main truss span (PP15 to PP19 in Span 4), as shown on the key plan presented in Figure 2.10. Shown in Figure 2.11 are schematic plan views of the two instrumented panels which shows locations and orientation of all LVDTs installed. As shown, all measurements were made on the upstream side of the truss, and at each end of each panel. Note that each end of the panels, there is a deck joint, and the deck is continuous over the floorbeams at interior panel points.

![Figure 2.10 – Key plan showing the two four-span deck units selected for displacement measurements and their locations with respect to the truss](image-url)
Figure 2.11 – Schematic plan views of portion of deck at (a) truss back span and (b) truss main span, showing location and orientation of LVDTs installed to measure relative displacements between the deck and the truss/floor system. 

*Note sign convention as indicated*
The LVDTs were attached to a magnetic base which mounted to the truss vertical. A target consisting of a piece aluminum angle was attached to the deck using a concrete anchor. Figure 2.12 presents a photograph of a longitudinal LVDT installation.

![Typical LVDT installation](image)

**Figure 2.12** – Typical LVDT installation for measurement of relative longitudinal displacement between the truss and deck

2.4.8 Transverse LVDTs

Additional displacement sensors (LVDTs) were installed at four locations to measure the relative transverse displacement between the top of the floorbeam (at the floorbeam-truss connection) and the deck. These LVDTs were installed at two locations in each of the deck panels described above in Section 2.4.7. In each panel, one sensor was installed at the joint (the end of the panel) and one was installed at the center floorbeam (see Figure 2.11).

The LVDTs were attached to a light-gage steel frame which was clamped to the top flange of the floorbeam. A magnetic base was used as a target for the LVDT and was attached to the exterior stringer near its top flange. Figure 2.13 presents a photograph of a transverse LVDT installation.
Figure 2.13 – LVDT installation for measurement of relative transverse displacement between the truss and deck at deck joint at PP4 (looking south)

2.5 Data Acquisition

Two Campbell Scientific CR9000 data loggers were used for the collection of data during both the controlled-load testing and the long-term monitoring phases of this project. This logger is a high speed, multi-channel 16-bit data acquisition system and was configured with digital and analog filters to assure noise-free signals. Real-time data were viewed while on site by connecting the logger directly to a laptop computer. This was done to assure that all sensors were functioning properly. This configuration was also used during the controlled load testing when data collection was started and stopped manually using the laptop.

Both CR9000 data loggers were enclosed in weather-tight boxes located on the inspection walkway beneath the truss spans, at Piers 2 (for sensors in Spans 1 and 2) and Pier 3 (for sensors in Spans 3 and 4). Figure 2.14 contains a photograph of the weather-tight enclosure located at Pier 3. Figure 2.15 presents a photograph of the inside of the box. In addition to the CR9000 data logger, there were communications equipment and a power supply inside the box.
Remote communications with the data logger were established using wireless modems installed at each data logger. Data download was intended to be performed automatically via a server located in the ATLSS laboratory in Bethlehem, PA. This link was also to be used to upload new programs as needed. However, due to the weak cellular signal at the bridge, a robust communications link was never achieved. Therefore, manual data collection was required. Nevertheless, high-quality data were obtained.
2.6 Electrical Power

In order to run the data logger and sensors during the long-term portion of the project, a continuous 110VAC power supply was obtained connecting into the navigation lighting circuit at the top of Pier 3. A power line was run along the inspection walkway to the data logger at Pier 2. Periodically the GFCI circuit tripped resulting in a temporary outage in data collection until the circuit was manually reset. However, an ample length of high-quality data was obtained.
3.0 Test Program – Summary

The following sections discuss the controlled-load testing and remote monitoring that was conducted.

3.1 Controlled Load Testing

3.1.1 Test Truck

A series of controlled load tests were conducted using a test truck of known geometry and weight. The truck had three main axles and a fourth floating rear axle. The test truck was fully loaded with stone, and had a gross vehicle weight (GVW) of 72,840 pounds. The individual axles of the truck were weighed on scales at the loading facility immediately before departing for the bridge. Figure 3.1 contains a photograph of the truck used for the testing. Table 3.1 contains the weight at each axle. Table 3.2 provides the key dimensions of the test truck.

![Test truck image](image)

Figure 3.1 – Test truck used during controlled load tests

<table>
<thead>
<tr>
<th>Test Description</th>
<th>Rear Axle Type</th>
<th>Front Axle Load (lb)</th>
<th>Rear Axle Group Load (lb)</th>
<th>GVW (lb)</th>
<th>Date of Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>Controlled Load Tests</td>
<td>Tandem²</td>
<td>12,920</td>
<td>59,920</td>
<td>72,840</td>
<td>November 19, 2005</td>
</tr>
</tbody>
</table>

Table 3.1 – Test truck axle load data

Note:
1. GVW = Gross Vehicle Weight
2. Floating third rear axle was in the up position for all tests except for test #1.
3.1.2 Testing Procedure and Summary

The controlled load tests were conducted between 11 AM and 2 PM on Monday November 21, 2005. It was a sunny day with a temperature of approximately 50 degrees F. Both travel lanes of the bridge were closed selectively during the passage of the test truck so only the response of the bridge to the test truck was recorded. Both lanes were reopened between tests to clear the queued traffic.

The tests consisted of a series of crawl and dynamic tests. For the crawl tests, the test truck was driven completely across the bridge between 3 and 5 mph. The dynamic tests were conducted with the test truck traveling at normal driving speed, which varied between 40 and 45 mph.

The crawl tests were conducted first. Each test was repeated three times and performed in both the southbound and northbound lanes. These tests were followed by the dynamic tests. A summary of the controlled load test data is presented in Table 3.3. As shown, a total of twelve tests were performed. The tests shaded in gray are those that were considered for data analysis. The tests performed more than once were reviewed to assess the repeatability of the data. The data were found to be repeatable. Therefore, one test (of each type) was selected for all further data analysis. Therefore, all tabulated data and charts presented in the remainder of this report are from one of these selected data files (shaded in gray in Table 3.3).
3.2 Remote Monitoring

The two CR9000 data loggers were also used for the long-term monitoring phase of this project. The bridge (at both locations) was monitored between November 11, 2005 and January 10, 2006. However, there were a number of power outages which occurred when the GFCI circuit at the power supply tripped. However, a total of 32 days of high-quality data were collected at both data loggers.

During the long-term monitoring, stress time-history data were not collected continuously. Data were only recorded when the measured stress at selected gages exceeded predefined triggers. The trigger gage and trigger value are selected solely to reduce the amount of time-history data recorded during the monitoring period. Generally by selecting a trigger value of stress equal to or slightly greater than the stress caused by the test truck, data will only be recorded during the passage of the heaviest trucks. These data can be used to validate the highest stress cycles recorded in the stress range histogram (which is recorded constantly over the monitoring period). Once the strain value for the “trigger” gage reached the predefined limit, the logger began recording data for a predefined period of time (equal to 40 seconds for this project). It should be noted that the trigger value of stress is not meant to be correlated to a stress caused by a particular vehicle. The value is selected so an appropriate quantity of data are recorded.

Each data logger operated independently. Therefore, each data logger was programmed with a unique trigger sensor and threshold value. When the measured strain at any one trigger gage was exceeded, data from all strain gages connected to the data logger were recorded. Therefore, there were two sets of long-term time-history data obtained (i.e., one for each data logger).

As noted, stress-range histograms were developed continuously at each location monitored using the rainflow cycle-counting method. For each strain gage, this method considers 10 minutes of time-history data at a time and pairs up peaks in the response in this 10 minute segment to determine a tally of stress range cycles (number and
magnitude). Every 10 minutes, the “tally” is updated, while the time-history data used to develop the “tally” is discarded. This results in significantly less data than a continuous recording time-history data. This process continued for the duration of the long-term monitoring period. Using these histograms, estimates of the effective stress-range and number of cycles can be made. Utilizing these results and knowing the detail category at the sensor location, and making the assumption that the stresses measured during the monitoring period are representative of the life of the bridge, an estimate of the remaining fatigue life can be made. A complete description of this procedure including a description of the rainflow cycle-counting algorithm is presented in Appendix B. Results of the long-term monitoring are presented in Section 5.
4.0 Results of Controlled-load Testing

The results of the controlled static and dynamic load tests are discussed in this section. Specifically, the global and local behavior of the bridge is examined utilizing data collected during the controlled load tests. Note that a “static” test refers to a test conducted with the test truck traveling at crawl speed, while a “dynamic” test refers to a test conducted with the test truck traveling at normal traveling speed.

A number of strain gages were installed in both the truss and approach spans to evaluate the global response of the bridge. These include strain gages on top and bottom flanges of girders and floorbeams, and on various truss elements such as chords, diagonals, and verticals. Detailed instrumentation plans are provided in Appendix A.

The response of the truss spans and approach spans will be presented and discussed separately below.

4.1 Truss Spans

4.1.1 Global Response

Strain gages were installed on a number of truss elements, and were generally concentrated on the upstream truss. Specifically, strain gages were installed the upper chord at midspan (U20-U21), lower chord at midspan (L20-L21), a hanger at midspan (U21-L21), and a diagonal at Pier 3 (U11-L12). Additionally, strain gages were installed on the downstream lower chord at midspan (L20-L21). Two strain gages were installed on each of these truss elements. Axial stresses in each member were calculated by taking the average of the two strain gages. Finally, strain gages were installed on the top and bottom flange of midspan of Floorbeam FB4 at midspan.

Shown in Figure 4.1 is a stress time-history for the upstream truss members and Floorbeam FB4 during the passage of the test truck in the northbound lane. As can be seen, the peak measured stresses in the truss members are low, equal to 0.5 ksi in the bottom chord, -0.7 ksi in the top chord, 1.0 ksi in the hanger, and 0.5 ksi in the diagonal. In the floor beam, the peak stresses were 1.8 ksi at the bottom flange, and -1.4 ksi at the top flange.

Upon examination of the top chord history, it can be seen that the response of the truss is as expected for a three span continuous structure. That is, when the test truck is in the center span, compression (negative) stresses in the top chord result. However, when the test truck is in the side spans, tension (positive) stresses result. This response can also be seen in the diagonal. The hanger on the other hand only responds to the test truck when it is in the vicinity of the hanger, as expected.

Figure 4.2 contains a similar plot for the load test with the test truck traveling southbound. The response is similar to the northbound test presented in Figure 4.1. It can be seen that the response of the top and bottom chords, and the floor beam is not reduced significantly. However, in the hanger and diagonal, the measured stresses are significantly less.
Figure 4.1 – Stress time-history plot for instrumented truss elements and floor beam for crawl test with test truck traveling northbound.
Figure 4.2 – Stress time-history plot for instrumented truss elements and floor beam for crawl test with test truck traveling *southbound*
4.1.2 Relative Deck Displacements

Shown in Figure 4.3 is a displacement time-history plot for the LVDTs located on either side of the deck panel between panel points 15 (CH_15) and 19 (CH_16) with the test truck in the northbound lane. As described above, each displacement measurement represents the relative longitudinal displacement between the deck and the truss. It can be seen that the response is a combination of global and local response. The global response is always present, but the local response is only evident when the test truck is directly over the instrumented locations. Therefore, when the test truck is some distance from panel points 15 and 19 (e.g., time less than 60 seconds and time greater than in Figure 4.3). This global response is the result of global deformation of the truss caused by the test truck. As noted in Figure 2.10, the positive direction for the measured displacements at CH_15 and CH_16 is 180 degrees opposite. Therefore, it can be seen that under global response, the deck panel is moving uniformly in one direction (since CH_15 and CH_16 are equal and opposite in sign) approximately 2 mils.

The local effects are predominant in the response when the test truck is in the vicinity of the displacement sensors (e.g., between time equal to 80 and 120 sec.). The presence of individual axles can be seen in the response. Furthermore, it appears that the local response is superimposed on the global response of the truss. For the duration of the test, the relative displacements between the deck and the truss are very small (less than 4 mils).

Finally, at the end of the test, there appears to be a very small (1 mil) offset in the response. It seems that the bridge does not return to its original position, though it must be noted that this is an extremely small displacement. Furthermore, this type of response which does not return to the original zero point is not uncommon in structural testing, and could be the result of a number of effects.
Figure 4.3 – Displacement time-history for longitudinal LVDTs installed at PP15 (CH_15) and PP19 (CH_16) for crawl test with test truck traveling northbound.
Figure 4.4 presents the displacement time-history plots for the two longitudinal displacement sensors at the either end of the deck panel between panel points 4 (CH_18) and 8 (CH_17) with the test truck in the northbound lane. As with the deck panel within the center truss span, the response appears to be the result of a combination of global and local effects.

The measured relative longitudinal displacements between the deck and truss are larger than for deck panel 15-19, however, they are still small at less than 10 mils. The results presented in Figures 4.3 and 4.4 are to be used as input into a finite element model of the as-built condition of the bridge. By calibrating this model using these measurements, more accurate predictions can be made of the response of the bridge after modifications have been made to the deck systems, such as making the deck continuous.

Figure 4.4 – Displacement time-history for longitudinal LVDTs installed at PP4 (CH_18) and PP8 (CH_17) for crawl test with test truck traveling northbound
Figure 4.5 presents a displacement time-history for the relative transverse displacement between the deck and the top flange of Floorbeam FB4 (CH_22) and Floorbeam FB6 (CH_21). Unlike the relative longitudinal displacements presented above, the transverse displacement appears to be the result of primarily local presence of the test truck. However, it can be seen that there is a global component of the response. Floorbeam FB6 is an interior floorbeam, while FB4 is at the expansion joint. Interestingly, the displacements are higher at the expansion joint despite the presence of the diaphragms there. As seen, the peak relative transverse displacement was on the order of 20 mils.

Figure 4.5 – Displacement time-history for transverse LVDTs installed at PP4 (CH_22) and PP6 (CH_21) for crawl test with test truck traveling northbound
4.1.3 Floorbeam Response

Shown in Figure 4.6 is a stress time-history plot for the two strain gages located at top and bottom flange of Floorbeam FB4 at midspan with the test truck in the northbound lane. The response of the floorbeam is as expected. The stress in the bottom flange is slightly higher than that in the top flange (there is a small contribution from the stringers and deck). Peak stress in the bottom flange is on the order of 1.8 ksi. Based on the average speed of the truck over the length of the bridge, and given the floorbeam spacing of 31 feet, the floorbeams were added to Figure 4.6 approximately 5 seconds apart. It should be noted that as with previous figures, the stresses in strain gage CH_15 do not return to zero. Again, this is not an uncommon occurrence in field testing and can be attributed to a number of causes.

Figure 4.6 – Stress time-history for strain gages installed on top (CH_15) and bottom flange (CH_16) of Floorbeam FB4 at midspan for crawl test with test truck traveling northbound
Figure 4.7 presents a stress time-history for the two strain gages installed at the top flange of Floorbeam adjacent to the connection to the east truss. The floorbeam flanges are welded to an endplate with full penetration welds which is in turn bolted riveted to the truss vertical. Two strain gages (CH_13 and CH_14) were installed directly adjacent to the full-penetration weld. Note that these gages were installed as part of the deck continuity study (along with the LVDT’s discussed previously). However, the stresses measured at these strain gages will also be used to assess the fatigue performance of this full-penetration weld.

Tensile stresses can be induced in the top flange as a result of in-plane bending of the floorbeam since the truss offers some rotational restraint (i.e., the floorbeam is not perfectly pinned). These stresses can be quantified by taking the average of the stresses at the two strain gages, indicated by the green line in the Figure.

Bending stresses can also be induced into the top flange as a result of out-of-plane bending of the floorbeam caused by relative longitudinal deformations between the deck and truss described above. These stresses can be quantified by taking half the difference between the measured stress at the two strain gages. This is indicated by the pink line in the Figure.

In general the response is complicated, and is again the result of global and local effects. However, the peak measured stress at the weld is very low (i.e., less than 0.6 ksi). These results can also be used to calibrate the computer analysis of the bridge.
Figure 4.7 – Stress time-history for strain gages installed on the top flange of Floorbeam FB4 adjacent to floorbeam/truss connection for crawl test with test truck traveling northbound
4.2 Approach Spans 1 and 2

4.2.1 Global Response

Strain gages were installed in Span 1 at the top (CH_11) and bottom (CH_12) of Girder G2 at midspan, and the top (CH_13) and bottom (CH_14) of Girder G1 at midspan.

Shown in Figure 4.8 is a stress time-history for each of these strain gages during the passage of the test truck in the northbound lane. Figure 4.9 contains the corresponding stress time-history during the passage of the test truck in the southbound lane. It can be seen that there is composite action between the girder and slab demonstrated by the low stresses measured in the top flange (CH_11, CH_13). Note that since the truck was traveling at a near constant speed, each of the plots shown essentially represents an influence line for the corresponding instrumented location for this truck. Furthermore, the measured stresses in the girders are comparable with the test truck in different lanes, i.e., with the test truck over Girder G1, the peak stress in Girder G1 was 2.25 ksi (Figure 4.8), and with the test truck over Girder G2, the peak stress in Girder G2 was the same value.

Figure 4.8 – Stress time-history plot at global strain gages installed on Girder G1 and G2 in Spans 1 and 2 for crawl test with test truck traveling northbound
Figure 4.9 – Stress time-history plot at global strain gages installed on Girder G1 and G2 in Spans 1 and 2 for crawl test with test truck traveling **southbound**
Strain gages were installed on the bottom flange of Stringers S1 (CH_9) and S4 (CH_10) at midspan between Floorbeams FB7 and FB8. Stress time-histories for these two strain gages are presented in Figures 4.10 and 4.11 for the passage of the test truck in the northbound and southbound lanes, respectively.

There is a superposition of global and local response. Since the stringers are spliced for the length of the bridge, the stringer responds to the presence of the test truck anywhere within Spans 1 and 2. The local presence of the truck axles can be seen in the strain gage directly under the path of the truck (i.e., CH_10 for the northbound test, and CH_9 for the southbound test). This effect is significantly reduced in the stringer not under the truck path.

Figure 4.10 – Stress time-history plot at strain gages installed on bottom flange of Stringer S1 (CH_9) and Stringer S4 (CH_10) in Span 2 at midspan between Floorbeams FB7 and FB8 for crawl test with test truck traveling northbound
Figure 4.11 – Stress time-history plot at strain gages installed on bottom flange of Stringer S1 (CH_9) and Stringer S4 (CH_10) in Spans 1 and 2 at midspan between Floorbeams FB7 and FB8 for crawl test with test truck traveling southbound.
4.2.2 Termination of Longitudinal Stiffeners

Cracks had previously been found at the termination of longitudinal stiffeners on the main girders (see Figure 2.6). To assess the cause of these cracks and the likelihood of cracks occurring at similar locations, back-to-back strain gages were installed on either side of the girder web at the termination of the stiffener-to-web weld. Two uncracked locations in Span 1 were selected. Figure 4.12 presents the stress time history for the two gages at one location: Girder G1 between FB4 and FB5. It can be seen that when the test truck is in Span 2 (away from the instrumented location), the two gages have nearly identical response, as would be expected. However, when the test truck passes directly overhead, it can be seen that superimposed on top of the global girder response is a local response which consists of out-of-plane bending of the web. This is characterized by the two strain gages responding in opposite directions. The magnitude of this out-of-plane bending stress is approximately 0.25 ksi (= (0.5 – 0.0)/2). Overall, the stresses appear to be of low magnitude as compared to the CAFL of 4.5 ksi for Category E.

![Stress time-history plot at strain gages installed at termination of longitudinal stiffener on Girder G1 in Span 1 on exterior (CH_1) and interior (CH_2) of girder web for crawl test with test truck traveling southbound](image-url)
4.2.3 Floorbeam Web Cope

Cracks had also been found propagating out of the floorbeam web cope at the connection to the main girders (see Figure 2.7). These cracks have been termed Type “T” cracks. Back-to-back strain gages were installed to measure the stresses directly adjacent to the cope radius at two locations: Floorbeam FB9 at Girder G1 (CH_15 and CH_16) and FB10 at G2 (CH_23 and CH_24). Figure 4.13 presents the stress time-history plot for these strain gages with the test truck in the northbound lane. There appears to be a global and local component of the response here as well. The peak stress of 6 ksi was measured at the end of FB9 (CH_15). The response appears to be dominated by in-plane bending of the floorbeam. There is an approximately 1 ksi (= (6.0 – 4.1)/2) out-of-plane bending stress however.

![Figure 4.13 – Stress time-history plot for strain gages installed back-to-back on web of Floorbeams FB9 (CH_15, 16, east end) and FB10 (CH_23, 24, west end) adjacent to web cope for crawl test with test truck traveling northbound](image-url)
4.2.4 Gusset Plate Termination

Strain gages were also installed back-to-back at the termination of the gusset-plate-to-girder-web weld (see Figure 2.8). Two such locations were selected for a total of four strain gages: the east ends of Floorbeam FB9 (CH_17, 18) and FB10 (CH_19, 20). Figure 4.13 presents the stress-time history for these four strain gages with the test truck in the southbound lane. It can be seen that the shape of the response for the two locations is similar however, the magnitudes are different. Note that the peak stresses for strain gages CH_17 and CH_18 are 3.0 ksi and 1.3 ksi, respectively. This indicates that there is a 2.2 ksi (= (3.0 + 1.3)/2) axial stress component, and a 0.9 ksi (= (3.0 – 1.3)/2) bending stress component. Therefore, there is a significant out-of-plane bending of the girder web at the gusset connection, since the component of out-of-plane bending (0.9 ksi) is large compared to the in-plane stress component. Overall however, the existence of out-of-plane bending stresses of such a magnitude is not surprising for a two-girder bridge based on past experience. Furthermore, as will be discussed in Section 5, the stresses measured at these details during the long-term monitoring phase of this project were low, indicating that they do not present a concern from a fatigue perspective.

![Stress time-history plot for strain gages installed back-to-back on web of Girder G1 at termination of gusset plate at Floorbeam FB9 (CH_17, 18) and FB10 (CH_19, 20) for crawl test with test truck traveling southbound](image-url)
4.2.5 **Floorbeam Knee Brace**

To assess the cause of cracks found in the knee brace to floorbeam bottom flange weld (termed a Type “K” crack), a single vertically oriented strain gage was installed on the floorbeam knee braces at each end of Floorbeam FB10 (see Figure 2.7). Strain gage CH_22 was installed at the east connection and CH_21 at the west connection. Shown in Figure 4.14 is a stress-time history for these two strain gages with the test truck in the northbound lane. As expected, the highest stresses are recorded in the east connection, however it should be noted that the stresses are tensile, resulting from the bending of the floorbeam under loading from a truck in one lane. It is likely that there is significant out-of-plane bending of the knee brace plate. With only one strain gage however, it is impossible to know with certainty. More importantly however, the measured stress is low in the knee brace (less than 0.5 ksi) compared to the CAFL for Category E of 4.5 ksi.

![Stress time-history plot for strain gages installed back-to-back on web of Girder G1 at termination of gusset plate at Floorbeam FB9 (CH_17, 18) and FB10 (CH_19, 20) for crawl test with test truck traveling northbound.](image-url)
4.3 Peak Measured Stresses

4.3.1 Truss Spans

Table 4.1 contains a summary of the maximum stress, minimum stress, and maximum stress range recorded at each sensor installed in the truss spans for crawl tests in each lane. Note that the fatigue evaluation of all details is presented in Section 5. Furthermore, the stress (or displacement) ranges presented are for the passage of a single truck (the test truck) in any given lane. These ranges do not represent the maximum ranges experience during the bridge which most likely would be caused by the passage of multiple trucks in either the same lane or different lanes.
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<td>-0.04</td>
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Table 4.1 – Summary of maximum, minimum, and peak stress (or displacement) range ($S_R = \text{max} - \text{min}$) measured during static tests for all lanes for strain gages located in the Truss Span.
The peak stresses at each strain gage were also determined for each of the dynamic tests. Shown in Table 4.2 is a summary of the ratio of measured peak dynamic stresses (maximum, minimum and stress range) to static stresses for all strain gages in the Truss Spans (the static results are presented above in Table 4.1). As can be seen, for the strain gages shown, the maximum dynamic amplification on the static stress ranges was 39% at strain gage CH_13 (top flange of Floorbeam FB4 at connection to upstream truss), however it should be noted that the stress magnitudes were low at this location. In the chord members of the truss (CH_1, CH_2, CH_5, CH_6, CH_7, and CH_8), the maximum stress range amplification was low at 12%. The amplification in the hanger (CH_3, CH_4) was also low at 13%. The amplification in the diagonal (CH_9, CH_10) was very low, equal to 4%. Finally, at the midspan of Floorbeam FB4, the amplification was 29%.
### Field Testing and Fatigue Evaluation of the Shippingport Bridge

#### FINAL REPORT

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<td>$S_{\text{min dynamic}}/S_{\text{min static}}$</td>
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Table 4.2 – Summary of dynamic-to-static stress ratios (maximum, minimum, and stress range) for each lane for strain gages located in the **Truss Spans**

44
4.3.2 Approach Spans 1 and 2

Table 4.3 contains a summary of the maximum stress, minimum stress, and maximum stress range recorded at each strain gage in Spans 1 and 2 for crawl tests in each lane. Note that the fatigue evaluation of all details is presented in Section 5. Furthermore, the stress ranges presented are for the passage of a single truck (the test truck) in any given lane. These ranges do not represent the maximum ranges experienced during the bridge which most likely would be caused by the passage of multiple trucks in either the same lane or different lanes.
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<td>Bottom flange Stringer S4 at midspan between FB7 and FB8</td>
<td>ksi</td>
<td>2.46</td>
<td>-0.21</td>
</tr>
<tr>
<td>CH_11</td>
<td>S.G.</td>
<td>Top flange G2 midspan Span 2</td>
<td>ksi</td>
<td>0.10</td>
<td>-0.24</td>
</tr>
<tr>
<td>CH_12</td>
<td>S.G.</td>
<td>Bottom flange G2 midspan Span 2</td>
<td>ksi</td>
<td>2.25</td>
<td>-0.62</td>
</tr>
<tr>
<td>CH_13</td>
<td>S.G.</td>
<td>Top flange G1 midspan Span 2</td>
<td>ksi</td>
<td>0.05</td>
<td>-0.17</td>
</tr>
<tr>
<td>CH_14</td>
<td>S.G.</td>
<td>Bottom flange G1 midspan Span 2</td>
<td>ksi</td>
<td>1.25</td>
<td>-0.46</td>
</tr>
<tr>
<td>CH_15</td>
<td>S.G.</td>
<td>FB9 web cope at G1 (south face)</td>
<td>ksi</td>
<td>6.12</td>
<td>-0.23</td>
</tr>
<tr>
<td>CH_16</td>
<td>S.G.</td>
<td>FB9 web cope at G1 (north face)</td>
<td>ksi</td>
<td>4.25</td>
<td>-0.21</td>
</tr>
<tr>
<td>CH_17</td>
<td>S.G.</td>
<td>South termination of gusset plate on G1 web at FB9 (inside)</td>
<td>ksi</td>
<td>1.76</td>
<td>-0.53</td>
</tr>
<tr>
<td>CH_18</td>
<td>S.G.</td>
<td>South termination of gusset plate on G1 web at FB9 (outside)</td>
<td>ksi</td>
<td>0.83</td>
<td>-0.25</td>
</tr>
<tr>
<td>CH_19</td>
<td>S.G.</td>
<td>South termination of gusset plate on G1 web at FB10 (inside)</td>
<td>ksi</td>
<td>1.49</td>
<td>-0.37</td>
</tr>
<tr>
<td>CH_20</td>
<td>S.G.</td>
<td>South termination of gusset plate on G1 web at FB10 (outside)</td>
<td>ksi</td>
<td>0.63</td>
<td>-0.19</td>
</tr>
<tr>
<td>CH_21</td>
<td>S.G.</td>
<td>Knee brace plate at FB10 and G1</td>
<td>ksi</td>
<td>0.04</td>
<td>-0.19</td>
</tr>
<tr>
<td>CH_22</td>
<td>S.G.</td>
<td>Knee brace plate at FB10 and G2</td>
<td>ksi</td>
<td>0.45</td>
<td>-0.09</td>
</tr>
<tr>
<td>CH_23</td>
<td>S.G.</td>
<td>FB10 web cope and G2 (south face)</td>
<td>ksi</td>
<td>2.82</td>
<td>-0.44</td>
</tr>
<tr>
<td>CH_24</td>
<td>S.G.</td>
<td>FB10 web cope at G2 (north face)</td>
<td>ksi</td>
<td>3.91</td>
<td>-0.51</td>
</tr>
</tbody>
</table>

Table 4.3 – Summary of maximum, minimum, and peak stress range ($S_R = \text{max} - \text{min}$) measured during static tests for all lanes for strain gages located in the Approach Spans
The peak stresses at each strain gage were also determined for each of the dynamic tests. Table 4.4 contains a summary of the ratio of measured peak dynamic stresses (maximum, minimum, and stress range) to static stresses for strain gages in the approach spans (static results are presented above in Table 4.3). Note that only results for strain gages with a static stress range greater than 0.5 ksi are presented.

The peak dynamic amplification of the static stress range in the main girders was 15% (CH_14 northbound). In the stringers, the amplification was higher at 45% (CH_9 northbound). A high amplification was also recorded at CH_22 (knee brace at FB10). However, the magnitude of the stresses at this location were low (peak dynamic stress of 1 ksi).
<table>
<thead>
<tr>
<th>Channel</th>
<th>Sensor</th>
<th>Location</th>
<th>Northbound</th>
<th>Southbound</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$S_{\text{max dynamic}}$</td>
<td>$S_{\text{min dynamic}}$</td>
</tr>
<tr>
<td>CH_1</td>
<td>S.G.</td>
<td>Termination of longitudinal stiffener, G1, Span 1, outside</td>
<td>1.17</td>
<td>1.45</td>
</tr>
<tr>
<td>CH_2</td>
<td>S.G.</td>
<td>Termination of longitudinal stiffener, G1, Span 1, inside</td>
<td>1.04</td>
<td>1.08</td>
</tr>
<tr>
<td>CH_3</td>
<td>S.G.</td>
<td>Termination of longitudinal stiffener, G2, Span 1, outside</td>
<td>0.73</td>
<td>0.96</td>
</tr>
<tr>
<td>CH_8</td>
<td>S.G.</td>
<td>Web gap at FB7 and G2</td>
<td>1.17</td>
<td>0.29</td>
</tr>
<tr>
<td>CH_9</td>
<td>S.G.</td>
<td>Bottom flange Stringer S1 at midspan between FB7 and FB8</td>
<td>1.56</td>
<td>0.90</td>
</tr>
<tr>
<td>CH_10</td>
<td>S.G.</td>
<td>Bottom flange Stringer S4 at midspan between FB7 and FB8</td>
<td>0.95</td>
<td>1.01</td>
</tr>
<tr>
<td>CH_12</td>
<td>S.G.</td>
<td>Bottom flange G2 midspan Span 2</td>
<td>0.86</td>
<td>1.04</td>
</tr>
<tr>
<td>CH_14</td>
<td>S.G.</td>
<td>Bottom flange G1 midspan Span 2</td>
<td>1.16</td>
<td>1.10</td>
</tr>
<tr>
<td>CH_15</td>
<td>S.G.</td>
<td>FB9 web cope at G1 (south face)</td>
<td>1.02</td>
<td>4.96</td>
</tr>
<tr>
<td>CH_16</td>
<td>S.G.</td>
<td>FB9 web cope at G1 (north face)</td>
<td>0.94</td>
<td>2.85</td>
</tr>
<tr>
<td>CH_17</td>
<td>S.G.</td>
<td>South termination of gusset plate on G1 web at FB9 (inside)</td>
<td>1.14</td>
<td>1.06</td>
</tr>
<tr>
<td>CH_18</td>
<td>S.G.</td>
<td>South termination of gusset plate on G1 web at FB9 (outside)</td>
<td>1.21</td>
<td>1.32</td>
</tr>
<tr>
<td>CH_19</td>
<td>S.G.</td>
<td>South termination of gusset plate on G1 web at FB10 (inside)</td>
<td>1.03</td>
<td>2.51</td>
</tr>
<tr>
<td>CH_20</td>
<td>S.G.</td>
<td>South termination of gusset plate on G1 web at FB10 (outside)</td>
<td>1.29</td>
<td>1.31</td>
</tr>
<tr>
<td>CH_22</td>
<td>S.G.</td>
<td>Knee brace plate at FB10 and G2</td>
<td>1.46</td>
<td>3.36</td>
</tr>
<tr>
<td>CH_23</td>
<td>S.G.</td>
<td>FB10 web cope and G2 (south face)</td>
<td>1.18</td>
<td>0.92</td>
</tr>
<tr>
<td>CH_24</td>
<td>S.G.</td>
<td>FB10 web cope at G2 (north face)</td>
<td>1.26</td>
<td>1.77</td>
</tr>
</tbody>
</table>

Table 4.4 – Summary of dynamic-to-static stress ratios (maximum, minimum, and stress range) for each lane for strain gages located in the **Approach Spans**
5.0 Results of Long-term Monitoring

This section of the report presents the results of the long-term monitoring phase of this project. Long-term monitoring was conducted between November 11, 2005 and January 10, 2006. However, there were a number of power outages which occurred when the GFCI circuit at the power supply tripped. However, a total of 32 days of high-quality data were collected at both data loggers.

Stress time-history data were recorded from all gages when predefined trigger values were exceeded in specified channels. Stress-range histograms were developed continuously for all channels throughout the monitoring period. Every ten minutes, histograms were updated for each channel and written to a file. The rainflow cycle-counting algorithm was used to develop the stress-range histograms. For the fatigue evaluation, the stress-range histograms were truncated at a level equal to approximately 1/4 of the constant amplitude fatigue limit (CAFL) of the detail specified in AASHTO. That is, all cycles with stress ranges less than the truncation level were removed from the histogram prior to calculation of the effective stress. An in-depth discussion of the methodology used for the fatigue evaluation can be found in Appendix B. Each detail type where monitoring was performed is analyzed and presented separately below.
5.1 Truss Spans

5.1.1 Global Strain Gages

Twelve strain gages were installed in the truss spans that were positioned to measure global response. Five truss members were instrumented, each with two strain gages (for a total of ten). In addition, strain gages were installed at the top and bottom flange of Floorbeam FB4 at midspan.

Table 5.1 contains the summary of the results of the fatigue evaluation of the global strain gages installed in the truss spans. For the riveted truss members, a fatigue category of D was used. The floorbeam is governed by fatigue Category C’, controlled by the welded transverse stiffener plates.

As shown in the Table, the CAFL was never exceeded at any of the locations for the duration of the monitoring. Therefore, infinite fatigue life can be expected for the governing details described above.

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>Location</th>
<th>Detail Cat.</th>
<th>$S_{R_{ eff}}$ (ksi)</th>
<th>$S_{R_{ max}}$ (ksi)</th>
<th>Cycles &gt; CAFL</th>
<th>Cycles/Week</th>
<th>Remaining Life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_1</td>
<td>Upstream top chord U20-U21 (east side)</td>
<td>D</td>
<td>1.8</td>
<td>2.0</td>
<td>0 0.00%</td>
<td>1.5</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_2</td>
<td>Upstream top chord U20-U21 (west side)</td>
<td>D</td>
<td>1.8</td>
<td>2.0</td>
<td>0 0.00%</td>
<td>4.4</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_3</td>
<td>Upstream hanger U21-L21 (east side)</td>
<td>D</td>
<td>*</td>
<td>1.0</td>
<td>0 0.00%</td>
<td>*</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_4</td>
<td>Upstream hanger U21-L21 (west side)</td>
<td>D</td>
<td>1.9</td>
<td>3.0</td>
<td>0 0.00%</td>
<td>125</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_5</td>
<td>Upstream bottom chord L20-L21 (east side)</td>
<td>D</td>
<td>-*</td>
<td>1.5</td>
<td>0 0.00%</td>
<td>*</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_6</td>
<td>Upstream bottom chord L20-L21 (west side)</td>
<td>D</td>
<td>-*</td>
<td>1.5</td>
<td>0 0.00%</td>
<td>*</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_7</td>
<td>Downstream bottom chord L20-L21 (east side)</td>
<td>D</td>
<td>1.8</td>
<td>2.0</td>
<td>0 0.00%</td>
<td>0.2</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_8</td>
<td>Downstream bottom chord L20-L21 (west side)</td>
<td>D</td>
<td>-*</td>
<td>1.5</td>
<td>0 0.00%</td>
<td>*</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_9</td>
<td>Upstream diagonal U11-L12 (east side)</td>
<td>D</td>
<td>-*</td>
<td>1.5</td>
<td>0 0.00%</td>
<td>*</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_10</td>
<td>Upstream diagonal U11-L12 (west side)</td>
<td>D</td>
<td>-*</td>
<td>1.5</td>
<td>0 0.00%</td>
<td>*</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_11</td>
<td>Top flange FB 4 at midspan</td>
<td>C’</td>
<td>-*</td>
<td>2.5</td>
<td>0 0.00%</td>
<td>*</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_12</td>
<td>Bottom flange FB4 at midspan</td>
<td>C’</td>
<td>3.3</td>
<td>3.5</td>
<td>0 0.00%</td>
<td>0.4</td>
<td>infinite</td>
</tr>
</tbody>
</table>

* all cycles less than the stress range cut-off of 1.5 ksi for Category D, and 3.0 ksi for Category C’

Table 5.1 – Summary of fatigue evaluation for global strain gages installed in **Truss Spans**
### 5.1.2 Floorbeam-to-Truss Connection

Presented in Table 5.2 are the results of the fatigue evaluation of the strain gages installed on the top flange of Floorbeam FB4 at the welded endplate connection to the upstream truss. This detail is considered as Category E.

As shown in the Table, the CAFL was never exceeded at any of the locations for the duration of the monitoring. In fact, at strain gage CH_13, the there were no stress-range cycles greater than the stress-range cutoff of 1 ksi for Category E. Therefore, infinite fatigue life can be expected for the governing details described above.

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>Location</th>
<th>Detail Cat.</th>
<th>$S_{R,eff}$ (ksi)</th>
<th>$S_{R,max}$ (ksi)</th>
<th>Cycles &gt; CAFL</th>
<th>Cycles/Week</th>
<th>Remaining Life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_13</td>
<td>Top flange FB4 at upstream</td>
<td>E</td>
<td>*</td>
<td>1.0</td>
<td>0</td>
<td>0*</td>
<td>infinite</td>
</tr>
<tr>
<td></td>
<td>truss (south edge)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CH_14</td>
<td>Top flange FB4 at upstream</td>
<td>E</td>
<td>1.3</td>
<td>1.5</td>
<td>0</td>
<td>5.2</td>
<td>infinite</td>
</tr>
<tr>
<td></td>
<td>truss (north edge)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* all cycles less than the stress range cut-off of 1.0 ksi for Category E.

Table 5.2 – Summary of fatigue evaluation for strain gages installed on the top flange of Floorbeam FB4 adjacent to the connection to the upstream truss
5.2 Approach Spans
5.2.1 Global Strain Gages

Six strain gages were installed within the approach spans to characterize the global behavior of these spans. Strain gages were installed on the top and bottom flange of Girders G1 and G2 at midspan of Span 2. Table 5.3 contains the summary of the results of the fatigue evaluation of the global strain gages installed in approach Spans 1 and 2. These strain gages are governed by Category E due to the gusset connection for the lateral bracing which is considered a long attachment. It should be noted however that this is conservative since the gusset detail is located some distance above the bottom flange (where the stresses were measured).

It was seen in the controlled load tests that the stress in the top flanges of the main girders was low due to composite action. This is also evident in the results of the long-term monitoring (see CH_11 and CH_13 in Table 5.3); the stress-range cutoff of 1 ksi was never exceeded during the 32 day monitoring at these gages.

At the bottom flanges however, the CAFL was exceeded with a frequency of greater than 0.01% (or 1 in 10,000). Therefore, finite life can be expected. However, the calculation indicates that greater than 100 years of remaining life is expected. Shown in Figure 5.1 is the stress-range histogram for the two strain gages at the bottom flanges of Girders G1 and G2.

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>Location</th>
<th>Detail Cat.</th>
<th>$S_{R,\text{eff}}$ (ksi)</th>
<th>$S_{R,\text{max}}$ (ksi)</th>
<th>Cycles $&gt;\text{CAFL}$</th>
<th>Cycles/Week</th>
<th>Remaining Life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_9</td>
<td>Stringer S1; midspan between FB7 &amp; FB8</td>
<td>C'</td>
<td>3.3</td>
<td>3.5</td>
<td>0</td>
<td>11.8</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_10</td>
<td>Stringer S4; midspan between FB7 &amp; FB8</td>
<td>C'</td>
<td>3.4</td>
<td>4.5</td>
<td>0</td>
<td>9.9</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_11</td>
<td>Top flange G2 midspan Span 2</td>
<td>E</td>
<td>-</td>
<td>1.0</td>
<td>0</td>
<td>0*</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_12</td>
<td>Bottom flange G2 midspan Span 2</td>
<td>E</td>
<td>2.0</td>
<td>6.0</td>
<td>24</td>
<td>1,670</td>
<td>over 100</td>
</tr>
<tr>
<td>CH_13</td>
<td>Top flange G1 midspan Span 2</td>
<td>E</td>
<td>-</td>
<td>1.0</td>
<td>0</td>
<td>0*</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_14</td>
<td>Bottom flange G1 midspan Span 2</td>
<td>E</td>
<td>2.4</td>
<td>5.0</td>
<td>5</td>
<td>1,410</td>
<td>over 100</td>
</tr>
</tbody>
</table>

* all cycles less than the stress range cut-off of 1.0 ksi for Category E.

Table 5.3 – Summary of fatigue evaluation for global strain gages installed in **Approach Spans**
Figure 5.1 – Stress-range histograms for strain gages located on the bottom flange of Girder G1 (CH_14) and Girder G2 (CH_12) at midspan of Span 1.
5.2.2 Longitudinal Stiffener Termination

Strain gages were also installed back-to-back at the termination of longitudinal stiffeners on the main girders in Span 2 at the bolted web splice. A crack had been found at a similar location in Span 1 during routine inspections. This detail is a Category E detail. Table 5.4 contains the results of the fatigue evaluation. As indicated, infinite life is expected.

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>Location</th>
<th>Detail Cat.</th>
<th>$S_{R,eff}$ (ksi)</th>
<th>$S_{R,max}$ (ksi)</th>
<th>Cycles &gt; CAFL</th>
<th>Cycles/Week</th>
<th>Remaining Life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_1</td>
<td>Back-to-back at longit. stiff. termination G1 (outside)</td>
<td>E</td>
<td>1.3</td>
<td>1.5</td>
<td>0</td>
<td>0.00%</td>
<td>0.9 infinite</td>
</tr>
<tr>
<td>CH_2</td>
<td>Back-to-back at longit. stiff. termination G1 (inside)</td>
<td>E</td>
<td>1.3</td>
<td>1.5</td>
<td>0</td>
<td>0.00%</td>
<td>4.1 infinite</td>
</tr>
<tr>
<td>CH_3</td>
<td>Back-to-back at longit. stiff. termination G2 (outside)</td>
<td>E</td>
<td>1.3</td>
<td>3.0</td>
<td>0</td>
<td>0.00%</td>
<td>326 infinite</td>
</tr>
<tr>
<td>CH_4</td>
<td>Back-to-back at longit. stiff. termination G2 (inside)</td>
<td>E</td>
<td>-*</td>
<td>1.0</td>
<td>0</td>
<td>0.00%</td>
<td>0* infinite</td>
</tr>
</tbody>
</table>

* all cycles less than the stress range cut-off of 1.0 ksi for Category E.

Table 5.4 – Summary of fatigue evaluation for strain gages installed back-to-back at termination of longitudinal stiffeners in Span 2
5.2.3 Floorbeam Web Cope

In Spans 1 and 2 the floorbeam webs near the top flange are coped with a very sharp transition at the connection to the main girders. Fatigue cracking (termed Type "T") has been found at this detail on this bridge. Back-to-back horizontal strain gages were installed adjacent to the cope at two uncracked locations. Table 5.5 presents the results of the fatigue evaluation for these strain gages. Fatigue Category E was considered. All stress-range cycles less than 1 ksi (equal to approximately CAFL/4) were removed from the spectra. As shown in the Table, at each strain gage the CAFL of 4.5 ksi for Category E was exceeded with a frequency greater than 0.01% (or 1 in 10,000). Therefore finite life can be expected. However, the calculations indicate that effectively infinite life can be expected at strain gages CH_15 and CH_16. The estimated remaining fatigue life at strain gage CH_24 is 61 years. Strain gage CH_23 was not functioning during the long-term monitoring phase of the project. Shown in Figure 5.2 is the stress-range histogram for these three strain gages.

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>Location</th>
<th>Detail</th>
<th>Cat.</th>
<th>$S_{R\text{eff}}$ (ksi)</th>
<th>$S_{R\text{max}}$ (ksi)</th>
<th>Cycles $&gt;$ CAFL</th>
<th>Cycles/Week</th>
<th>Remaining Life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_15</td>
<td>Floorbeam FB9 web cope at Girder G1 (south face)</td>
<td>E</td>
<td>3.2</td>
<td>9.0</td>
<td>1846</td>
<td>10.58%</td>
<td>3,750</td>
<td>over 100</td>
</tr>
<tr>
<td>CH_16</td>
<td>Floorbeam FB9 web cope at Girder G1 (north face)</td>
<td>E</td>
<td>2.6</td>
<td>7.0</td>
<td>296</td>
<td>2.58%</td>
<td>2,470</td>
<td>over 100</td>
</tr>
<tr>
<td>CH_24</td>
<td>Floorbeam FB10 web cope at Girder G2 (north face)</td>
<td>E</td>
<td>3.9</td>
<td>10.0</td>
<td>2550</td>
<td>14.79%</td>
<td>3,710</td>
<td>61</td>
</tr>
</tbody>
</table>

Table 5.5 – Summary of fatigue evaluation for strain gages installed at floorbeam web cope at connection to main girders
Figure 5.2 – Stress-range histograms for strain gages located at the web copes at Floorbeam FB9 at the east end (CH_15, CH_16) and west end (CH_24)
5.2.4 Gusset Plate Termination

Two gusset plate terminations in Span 1 were selected for instrumentation. At each location, two strain gages were installed back-to-back on either side of the girder web directly adjacent to the gusset plate termination. As a long attachment, fatigue Category E was considered. All stress-range cycles less than 1 ksi (equal to approximately CAFL/4) were removed from the spectra.

Table 5.6 contains the results of the fatigue evaluation utilizing the stress range-histograms measured at these four strain gages. At two of the strain gages, the CAFL was exceeded with a frequency of more than 0.01% (CH_17 and CH_19). However, the fatigue life calculations indicate that greater than 100 years of remaining life are expected. The CAFL was never exceeded at the other two strain gages (CH_18 and CH_20), and therefore infinite fatigue life is expected. Presented in Figure 5.3 is the stress-range histograms for these four strain gages.

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>Location</th>
<th>Detail Cat.</th>
<th>$S_{R, eff}$ (ksi)</th>
<th>$S_{R, max}$ (ksi)</th>
<th>Cycles $&gt; \text{CAFL}$</th>
<th>Cycles/Week</th>
<th>Remaining Life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_17</td>
<td>South termination of gusset plate on Girder G1 web at FB9 (inside)</td>
<td>E</td>
<td>2.7</td>
<td>7.0</td>
<td>168</td>
<td>1.77%</td>
<td>2,040</td>
</tr>
<tr>
<td>CH_18</td>
<td>South termination of gusset plate on Girder G1 web at FB9 (outside)</td>
<td>E</td>
<td>1.7</td>
<td>3.0</td>
<td>0</td>
<td>0.00%</td>
<td>689</td>
</tr>
<tr>
<td>CH_19</td>
<td>South termination of gusset plate on Girder G1 web at FB10 (inside)</td>
<td>E</td>
<td>2.1</td>
<td>5.0</td>
<td>2</td>
<td>0.03%</td>
<td>1,280</td>
</tr>
<tr>
<td>CH_20</td>
<td>South termination of gusset plate on Girder G1 web at FB10 (outside)</td>
<td>E</td>
<td>1.7</td>
<td>3.0</td>
<td>0</td>
<td>0.00%</td>
<td>565</td>
</tr>
</tbody>
</table>

Table 5.6 – Summary of fatigue evaluation for strain gages installed back-to-back at termination of gusset plates in Span 2
Figure 5.3 – Stress-range histograms for strain gages located at gusset plate terminations at the west end of Floorbeam FB9 (CH_17, 18) and Floorbeam FB10 (CH_19, 20)
5.2.5 Floorbeam Knee Brace

A single vertically oriented strain gage was installed on the flange plate of two knee braces in Span 2 (each end of Floorbeam FB10). Termed Type “K” cracking, it has been found on this bridge during routine inspection. This detail is considered as fatigue Category E. All stress-range cycles less than 1 ksi (equal to approximately CAFL/4) were removed from the spectra.

Presented in Table 5.7 are the results of the fatigue evaluation. As shown, the CAFL of 4.5 ksi was never exceeded. The maximum stress range was 1.5 ksi. Therefore, infinite fatigue life can be expected.

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>Location</th>
<th>Detail Cat.</th>
<th>$S_{R,\text{eff}}$ (ksi)</th>
<th>$S_{R,\text{max}}$ (ksi)</th>
<th>Cycles $&gt; \text{CAFL}$</th>
<th>Cycles/Week</th>
<th>Remaining Life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_21</td>
<td>Knee brace plate at Floorbeam FB10 at Girder G1</td>
<td>E</td>
<td>-*</td>
<td>1.0</td>
<td>0</td>
<td>0*</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_22</td>
<td>Knee brace plate at Floorbeam FB10 at Girder G2</td>
<td>E</td>
<td>1.3</td>
<td>1.5</td>
<td>0</td>
<td>31</td>
<td>infinite</td>
</tr>
</tbody>
</table>

* all cycles less than the stress range cut-off of 1.0 ksi for Category E.

Table 5.7 – Summary of fatigue evaluation for strain gages installed back-to-back at termination of gusset plates in Span 2
5.2.6 Web Gaps

Within Spans 1 and 2, floorbeam connection plates were not welded to the tension flanges of the main girders. This can be a concern for distortion-induced fatigue cracking, depending on the magnitude of the out-of-plane stresses in the web gap. However, no cracked web gaps have been found on the bridge to date.

The web gaps were very small due to the small clip on the corner of the connection plate. This condition appeared to be common on the bridge. As a result, it was impossible to install the strip gages in the web gap itself. Therefore, the gages were installed adjacent to the web gap. Only one gage was functioning at each locations. Table 5.8 presents the results of the fatigue evaluation for these strain gages. Fatigue Category C was used. As shown, the CAFL was never exceeded and infinite life can be expected. In fact, there were no cycles even exceeding the stress-range cutoff of 2.5 ksi for Category C.

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>Location Details</th>
<th>Detail Cat.</th>
<th>$S_{R_{eff}}$ (ksi)</th>
<th>$S_{R_{max}}$ (ksi)</th>
<th>Cycles $&gt; CAFL$</th>
<th>Cycles/Week</th>
<th>Remaining Life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_5</td>
<td>Web gap at Floorbeam FB7 and Girder G1</td>
<td>C</td>
<td>*</td>
<td>0.5</td>
<td>0</td>
<td>0.00%</td>
<td>0* infinite</td>
</tr>
<tr>
<td>CH_8</td>
<td>Web gap at Floorbeam FB7 and Girder G2</td>
<td>C</td>
<td>*</td>
<td>2.0</td>
<td>0</td>
<td>0.00%</td>
<td>0* infinite</td>
</tr>
</tbody>
</table>

* all cycles less than the stress range cut-off of 2.5 ksi for Category C.

Table 5.8 – Summary of fatigue evaluation for strain gages installed at web gaps at floorbeam connection plates
6.0 **Conclusions**

Based on the results of controlled-load testing and long-term monitoring presented above, the following conclusions can be drawn:

**Truss Spans**

1. Peak stresses measured in the truss members and floorbeam during the controlled-load testing were generally very low, as summarized below:
   a. **Chords** – The peak measured static stress ranges in the instrumented top and bottom chords were **0.9 ksi** and **0.75 ksi**, respectively.
   b. **Hanger** – The peak measured static stress range in the instrumented hanger was **1.85 ksi**.
   c. **Diagonal** – The peak measured static stress range in the instrumented diagonal was **0.59 ksi**.
   d. **Floorbeam** – The peak measured static stress ranges in the instrumented floorbeam at midspan and at the connection to the truss were **1.88 ksi** and **0.78 ksi**, respectively.

2. The global response of the truss spans is generally as expected.

3. Relative longitudinal and transverse displacements of two deck panels were measured and can be used to calibrate computer analyses of the bridge. Increased confidence in the results of analyses considering fully continuous deck condition (a potential retrofit strategy to address deterioration of the floor system at existing deck joints) can be expected.

4. Amplification of stresses due to dynamic effects were measured, and are summarized as follows:
   a. **Chords** – The peak measured dynamic stress range was **1.14** times the peak measured static stress range.
   b. **Hanger** – The peak measured dynamic stress range was **1.13** times the peak measured static stress range.
   c. **Diagonal** – The peak measured dynamic stress range was **1.09** times the peak measured static stress range.
   d. **Floorbeam** – At midspan, the peak measured dynamic stress range was **1.29** times the peak measured static stress range. At the connection to the truss, the ratio was **1.39**.

5. The results of long-term monitoring indicate that fatigue does not appear to be a concern for the truss spans assuming traffic patterns observed during the monitoring period are reasonably representative of those in the past and those in the future. Low stress ranges were measured during long-term monitoring at the welded top flange-to-truss connection.
Approach Spans

6. Peak stresses measured in the girders and stringers during the controlled-load testing are summarized below:

   a. **Girders** – The peak measured static stress ranges at the bottom flange of Girders G1 and G2 at midspan of Span 2 were **2.78 ksi** and **2.87 ksi**.

   b. **Stringers** – The peak measured static stress ranges at the bottom flange of Stringers S1 and S4 at midspan between Floorbeams FB7 and FB8 were **2.75 ksi** and **2.67 ksi**, respectively.

7. The global response of Spans 1 and 2 is generally as expected.

8. Amplification of stresses due to dynamic effects were measured, and are summarized as follows:

   a. **Girders** – The peak measured dynamic stress range was **1.15** times the peak measured static stress range.

   b. **Stringers** – The peak measured dynamic stress range was **1.45** times the peak measured static stress range.

9. The results of long-term monitoring indicate that fatigue does not appear to be a significant concern for the approach spans assuming traffic patterns observed during the monitoring period are reasonably representative of those in the past and those in the future. Conclusions from the long-term monitoring of specific details are as follows:

   a. **Stringer diaphragm weld** – The stresses measured at the bottom flange of Stringers 1 and 4 at midspan indicate that fatigue is not a concern at this detail and infinite life can be expected.

   b. **Longitudinal stiffener termination** – Measurements made at strain gages installed at the termination of the longitudinal stiffener at web splices in Span 1 indicate that infinite fatigue life can be expected. The CAFL of 4.5 ksi was never exceeded at any of the strain gages (the peak measured stress range was 3 ksi).

   c. **Floorbeam web cope** – Several cracks have been found at the floorbeam web cope (termed Type “T” cracks in the inspection reports). Higher stresses were measured at these instrumented locations (peak stress range of 10 ksi). Furthermore, the CAFL of 4.5 ksi was frequency exceeded (14.79% of stress range cycles exceeded the CAFL at one location). However, a fatigue evaluation utilizing the measured stress-range histograms indicates that effectively infinite life can be expected at two locations. At the third location, 61 years of remaining life is predicted. It may be prudent to retrofit these locations considering the somewhat high stresses and the fact that the retrofit would be relatively easy to implement, consisting of a cored hole at the end of the flange termination, and grinding the hole edges to ensure a smooth stress flow.
d. **Gusset plate termination** – Fatigue life calculations based measured stress ranges at the gusset plate termination indicate that the CAFL is exceeded with a peak frequency of 1.77%, resulting in a finite life estimation. However, the life calculation indicates that greater than 100 years of remaining fatigue life (effectively infinite life) can be expected.

e. **Floorbeam knee brace** – A single crack had been found in the weld between the knee brace flange and the bottom flange of Floorbeam FB6 (termed a Type “K” crack in the inspection report). Based on the low stresses measured in the two instrumented knee brace flange plates, infinite fatigue life can be expected.

f. **Web gap at floorbeam connection plate** – At the web gaps instrumented, the stress ranges were low and infinite fatigue life is expected. The locations selected are expected to be reasonably representative of the type of web gap details found throughout this bridge. No cracks of this type have been found on the bridge to date.

10. It should be noted at many details, strain gages were not installed in a nominal stress field since the cracking at these details is driven by secondary (or local) bending of the connected elements. Therefore the stresses measured may represent somewhat of a localized or “hot spot” stress. However, use of such data to evaluate cracking resulting from secondary stresses is commonly performed. It can also be noted that despite measuring elevated stresses, the fatigue evaluation indicates that there is a generally low fatigue concern on these spans (see conclusion 9. above).
APPENDIX A

Instrumentation Plans
APPENDIX B

Development of Stress-range Histograms used to Calculate Fatigue Damage
B.1 Stress-Range Histograms

The stress-range histogram data collected during the uncontrolled monitoring permitted the development of a random variable-amplitude stress-range spectrum for the selected strain gages. It has been shown that a variable-amplitude stress-range spectrum can be represented by an equivalent constant-amplitude stress range equal to the cube root of the mean cube (rmc) of all stress ranges (i.e., Miner’s rule) [1] (i.e., \( S_{\text{reff}} = \left[ \sum \alpha_i S_{ri}^3 \right]^{1/3} \)).

During the long-term monitoring program, stress-range histograms were developed using the rainflow cycle counting method [2]. Although several other methods have been developed to convert a random-amplitude stress-range response into a stress-range histogram, the rainflow cycle counting method is widely used and accepted for use in most structures. During the long-term monitoring program, the rainflow analysis algorithm was programmed to ignore any stress range less than 0.50 ksi (18 με). Hence, the “raw” histograms do not include these very small cycles. Such small cycles do not contribute to the overall fatigue damage of even the worst details and if included, can actually unconservatively skew the results, as will be discussed below. It is also worth mentioning, that in some testing environments, the validity of stress-range cycles less than this are often questionable due to electromechanical noise.

The rainflow cycle counting method considers a fixed period (10 minutes was used for this project) of time-history data (i.e., stress versus time). First, the tensile and compressive peaks are determined. Then the peaks are paired up to determine the number and magnitude of stress range cycles which are totaled to form a stress-range histogram for that particular period of time. This process is repeated for the next segment of time. The histograms are summed in order to develop a cumulative stress-range histogram. It should be noted that since the peaks are paired up within a block of time (e.g., 10 min.), one stress cycle may not necessarily be the result of one vehicle. For instance if one truck causes tensile stress in a detail while crossing in the southbound lane, and a similar truck causes compressive stress at the same detail while crossing in the northbound lane (both crossings occur within the same 10 minute block of time), the stress range would be the peak-to-peak stress caused by the two trucks (assuming no other vehicles cross the bridge in this time period).

The effective stress range presented for each channel in the body of the report was calculated by ignoring all stress-range cycles obtained from the stress-range histograms that were less than predetermined limits. (It should be noted that the limit described here should not be confused with the limit described above. The limit above (i.e., 0.50 ksi (18με)) refers to the threshold of the smallest amplitude cycle that was counted by the algorithm and not related to the cycles that were counted, but later ignored, to ensure an accurate fatigue life estimate, as will be discussed.) For all welded steel details, a cut-off or threshold is appropriate and necessary, as will be discussed. The limits were typically about ¼ the constant amplitude fatigue limit for the respective detail. For example, for strain gages installed at details that are characterized as category C, with a CAFL of 10.0 ksi, the cutoff was set at 2.5 ksi. Hence, stress range cycles less than 2.5 ksi were ignored in the preparation of the stress-range histograms used to calculate the effective stress range and the number of cycles accumulated. The threshold was selected for two reasons.
Previous research has demonstrated that stress ranges less than about $\frac{1}{4}$ the CAFL have little effect on the cumulative damage at the detail [3]. It has also been demonstrated that as the number of random variable cycles of lower stress range levels are considered, the predicted cumulative damage provided by the calculated effective stress range becomes asymptotic to the applicable S-N curve. A similar approach of truncating cycles of low stress range is accepted by researchers and specifications throughout the world [4].

![Figure B.1 – Effect of truncating cycles at different stress range cut off levels](Typical data from a strain gage at a fatigue sensitive detail)

Figure B.1, shows the effect on the calculated effective stress range for several levels of truncation using typical field acquired long-term monitoring data collected from strain gage installed on a bridge. The data presented in Figure B.1 are also listed in Table B.1 showing the selected truncation level and its impact on the effective stress range.

As demonstrated by Figure B.1, as the truncation level decreases (from the lowest level), the effective stress range and corresponding number of cycles approaches the slope of the S-N curve for Category C, which is also plotted in Figure B.1 (i.e., a slope of $-3$ on a log-log plot). As long as the cut off level selected is consistent with the slope of the fatigue resistance curve, considering additional stress cycles at lower truncation levels does not improve the damage assessment and can therefore be ignored. As can be seen, using a truncation level as high as 10 ksi, the curve is nearly asymptotic to the slope of the S-N curves. Hence, an accurate prediction of the total fatigue life results.
It should also be noted that the load spectrum assumed in the AASHTO LRFD specifications for design was developed by only considering vehicles greater than about 20 kips [5]. Thus the AASHTO LRFD design also implicitly truncates and ignores stress cycles generated by lighter vehicles and vibration [6]. The observed frequency of stress cycles obtained from traffic counts is also consistent with the frequency of vehicles measured.
Table B.1 – Calculated effective stress ranges using different stress range cut off levels
Only every other data shown in Figure B.1 is shown for brevity

The maximum stress ranges listed in the tables developed in the body of this report were determined from the rainflow count. According to rainflow cycle counting procedures, the peak and valley that comprise the maximum stress range may not be the result of a single loading event and may in fact occur hours apart. In other words, an individual truck did not necessarily generate the maximum stress range shown in the tables. This is particularly true of distortion induced stresses that are subjected to reversals in stress due to eccentricity of the loading. In many cases, it was possible to identify this maximum stress range with a specific vehicle passage, but in other cases, the maximum rainflow stress range exceeded the maximum stress range from any individual vehicle. During the remote long-term monitoring program, the stress-range histograms were updated every ten minutes. Hence, the longest interval between nonconsecutive peaks and valleys is ten minutes.
B.2 Frequency of Exceedence of the CAFL

Based on experimental data, it has been found that when cycles in the variable amplitude spectrum exceed the CAFL often enough, then all stress cycles experienced by the structure can be considered to be damage-causing. This frequency of exceedence limit ranges between 0.01% and 0.05%. This corresponds to an occurrence of 1 in 10,000 or 1 in 2,000.

Research indicates that if this frequency limit is not exceeded, then it is reasonable to conclude that fatigue cracking would not be expected and infinite life can be assumed. However, if the limit is exceeded, the potential for fatigue cracking of the member exists and the fatigue life can be estimated by extending the given S-N curve. Obviously, this extension will only be required if the effective stress range (S_{Reff}) is less than the CAFL of the detail.

It should be noted that the limits are somewhat different for different details and the experimental data are limited. It is perhaps overly conservative to set the limit at 0.01% one for all details when conducting a fatigue evaluation. (This is not an issue in the design of new structures.) However, some owners may feel that 0.05% is too liberal and that a more conservative approach is best. Therefore, for the purposes of this study, a limit of 0.01% has been used.
References: