Field Testing and Fatigue Evaluation of the I-79 Bridge over the Ohio River

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FIELD TESTING AND FATIGUE EVALUATION OF THE I-79 NEVILLE ISLAND BRIDGE OVER THE OHIO RIVER

Final Report

by

Robert J. Connor
Ian C. Hodgson
Hussam Mahmoud
Carl Bowman

Prepared for:
Modjeski and Masters
PennDOT

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February 2005
FIELD TESTING AND FATIGUE EVALUATION OF THE I-79 NEVILLE ISLAND BRIDGE OVER THE OHIO RIVER

Final Report

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ATLSS Report No. 05-02

February 2005
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Executive Summary

The Neville Island Bridge complex carries interstate I-79 over the Ohio River, Neville Island, and the Ohio River backchannel, between Coraopolis and Sewickley, Pennsylvania. Opened to traffic in the mid-seventies, the bridge is actually a complex of both mainline spans as well as entrance and exit ramps. The main span over the Ohio River consists of a 750 foot tied arch span. Other spans consist of both straight and curved plate girders.

A complete fatigue and fracture evaluation of the bridge was performed to address problems identified during periodic inspections over recent years. Researchers at the ATLSS Center of Lehigh University were contracted by the firm of Modjeski and Masters at the direction of Penn DOT to conduct the study. Four portions of the bridge were selected for field instrumentation:

1. Ramp J - Two-span continuous, two curved steel plate girders
2. Tied Arch
3. Span 25, 26 – Part of four- and two-span continuous units, respectively; straight steel plate girders
4. Ramp A – Steel bent cap

Instrumentation plans were developed for each location and consisted of installation of weldable resistance strain gages at key locations both to understand the response of the bridge to load and to quantify the stress-range histograms at critical details. Displacement sensors were also installed at selected locations to measure secondary deformations at web gaps. Both controlled load tests using a test truck of known weight and geometry and long-term monitoring of random traffic were performed.

Estimates of the remaining fatigue life were made based on the stress-range histograms determined from the data. Retrofit solutions were recommended for locations where either the predicted life was insufficient, or where cracking has already been observed. In addition, retrofit recommendations were made to address the “Hoan-like” details present throughout the bridge complex.

General Observations

A considerable amount of data were collected and analyzed as part of this study. It is felt that some of the more general findings can likely be extended to other similar bridges and considered during additional evaluations of those bridges. Some of these are summarized below.

- Overall, the noncomposite spans behave fully composite under service loads. This finding is consistent with many other field studies conducted by the ATLSS team as well as others. This could be considered when performing a fatigue evaluation of other bridge designed as noncomposite if remaining life is marginal. Although field verifications are required, it is reasonable to make preliminary calculations assuming composite action to establish the potential payoff if such studies were conducted and were to reveal the structure actually behaves compositely.
The cause for cracks located at the welded connection between the top flange and transverse connection plates, identified as “K-cracking” in previous inspection reports, has been determined. The cause is the result of floorbeam flexibility and the incompatibility between the floorbeam and the deck slab. A limited number of other bridges have been observed to have a similar cracking problem. It is almost certain the cause of these cracks is due to the same driving mechanisms. The cracks are not necessarily a major concern as long as the connection is not completely severed. Once the connection completely cracks, out-of-plane distortion cracking is likely to precipitate. Bridges with long span floorbeam not directly attached to the concrete deck and where the top flange of the primary girders is embedded in the deck and should be considered to be the most susceptible systems.

Video monitoring coupled with stress-time history data revealed that a limited number of permitted super loads crossed the bridges during the monitoring period. Although these vehicles produce larger stress range cycles than all other individual trucks, the frequency of occurrence is very low. Hence, these trucks do not significantly contribute to the cumulative fatigue damage in the bridge. Obviously, if the number of cycles (i.e., permitted super loads) increased substantially, fatigue damage from these trucks could be a concern. However, for this bridge and others on this segment of I-79, it is unlikely that these super loads will produce significant fatigue damage. It is noted however that the strength limit state must be considered when allowing these trucks to pass, which is believed to be considered during the permitting process.
1.0 Project Summary and Background

1.1 Introduction

The Neville Island Bridge is located about 8.5 miles North West of downtown Pittsburgh, Pennsylvania and carries interstate I-79 over the Ohio River. The bridge was opened to traffic in the mid-seventies. The main line spans the Ohio River, Neville Island, and the Ohio River backchannel as well as railroad tracks and SR-65 on the northern shore of the river. An aerial photograph of a portion of the bridge complex is contained in Figure 1.1.

![Figure 1.1 – Aerial photograph of the Neville Island bridge complex]( Courtesy of teraserver-usa.com)

The bridge complex consists of a variety of bridge types and geometries which carry both the main line and various entrance and exit ramps. The main span over the Ohio River consists of a tied arch with a span of 750 feet. Other spans consist of both straight and curved welded steel plate girders. The bridge complex has been in service for almost 30 years.

During a prior fatigue study of the bridge, live load stress ranges at critical details were determined analytically and compared to allowable stress ranges per AASHTO. The results of this study indicated that a large number of locations required retrofitting. In order to reduce the number of required retrofits, it was decided that field measurements should be made to determine the actual stress ranges. It is well known that
field measured stresses can be significantly less than analytically determined stress ranges.

Lehigh University’s ATLSS Center was contracted by the firm of Modjeski and Masters to perform a detailed fatigue and fracture evaluation of selected portions of the bridge, as determined in collaboration with PennDOT District 11-0 and Modjeski and Masters. Encompassing various spans of the bridge, the scope of this work included estimation of the remaining fatigue life at previously identified critical details (including “Hoan-like” details) and developing retrofit strategies where deemed necessary.

In addition to the remaining fatigue life estimates, a fracture assessment was conducted on cores taken at some questionable details in Ramp J and the tied arch. The location of the extracted cores was determined in collaboration with the firm of SAI, PennDOT District 11-0 and Modjeski and Masters. Fractographic analysis and linear elastic fracture mechanics (LEFM) were used for the assessment of the cores. Detailed fracture evaluation assessment can be found in Appendix C.

1.2 Spans Selected for Investigation

Field instrumentation of four independent locations within the complex was performed. These locations consist of the following:

1. Ramp J, H - Two-span continuous, two curved steel plate girders
2. Span 25, 26 – Part of four- and two-span continuous units, respectively; straight steel plate girders
3. Tied Arch
4. Ramp B – Steel bent cap

At each of the four locations listed above, an instrumentation plan was developed. There were a number of fatigue sensitive details within each span type that had previously been identified. These details are listed below in Table 1.1.
Table 1.1 – Summary of fatigue sensitive details selected for instrumentation

Strain gages and displacement sensors were located to capture data necessary to allow researchers to understand the behavior of the bridge which is complex in some locations.

Long-term monitoring was also conducted to accurately estimate the effective stress ranges at critical details. Utilizing these data, an estimate of the remaining fatigue life can be made. It is important to mention that long-term monitoring was done only at locations 1, 2, and 3. Long-term monitoring at location 4 (Ramp B) was unnecessary since short term monitoring (2 days) revealed that the response of the bent cap at all strain gages to normal traffic was very low. Instrumentation layout and general response of bent cap can be found in Appendix E.

A data logger was placed at each location. The data logger at Ramps J and H was connected directly to the local area network (LAN) within the PennDOT office trailer on Neville Island (the office is directly beneath Ramp J). The data loggers at the tied arch and spans 25-26 were connected to the network utilizing a wireless networking system. The data loggers were connected to the internet via a broadband internet connection provided by PennDOT within the trailer.
1.3 Instrumentation and Data Acquisition

The following section describes the instrumentation used for the controlled load testing and long-term monitoring. Details on the location of the strain gages installed on the bridges can be found in Appendix A.

1.3.1 Strain Gages

For ease of installation, weldable uniaxial strain gages were used. The gages were type LWK-06-W250B-350 produced by Measurements Group Inc. with an active grid length of 0.25 inches. Grinding and cleaning was the only preparation needed for the metal surfaces before the installation of the gages. The gage itself is pre-bonded to a metal foil by the manufacturer. It is then spot welded to the structure in the field. After installation, gages were covered with multi-layer system then sealed with a silicon type agent.

The gages are uniaxial and temperature-compensated for use on structural steel. The gage resistance is 350Ω and an excitation voltage of 10 volts was used.

1.3.2 Displacement Sensors

Linear variable differential transformers (LVDTs), manufactured by Macro Sensors were used and were mounted to magnetic bases installed on the bridge. The sensors have a displacement range of ±1/4 inch and have infinite resolution. The resolution of the measurement is limited by the data acquisition system and was better than 1x10^-5 in. The sensors are encased in stainless steel housing and are suitable for use in harsh environments.

1.3.3 Data Acquisition

With the exception of Ramp B, Campbell Scientific CR9000 Data Loggers were used for the collection of the data throughout the controlled load testing. This logger is a high speed, multi-channel 16-bit system. The data loggers were configured with digital and analog filters to assure noise-free signals. Real-time data were viewed while on site by connecting a logger 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.

At Ramp B, a CR5000 data logger was used. This unit is also a 16-bit system however; it does not have digital filtering capability. However, noise in the data was minimized using on board digital integration techniques.

The data loggers were enclosed in weather-tight boxes. At the tied arch, the data logger was located within the east tie girder near the north pier. Figures 1.2 and 1.3 show the installation of the data acquisition system used at Ramp J. The installations at Spans 25 and 26 and Ramp B were similar.
Figure 1.2 – Data acquisition system used for the on-site controlled load testing, and Long-term monitoring at Ramp J
1.4 Remote Long-term Monitoring

CR9000’s were also used for the long term monitoring of the bridges. Stress-time history data were not collected continuously. Data were only recorded when the measured stress at selected gages exceed predefined triggers. Once the strain value for that gage reaches the limit defined, the logger starts recording data for all the gages on the bridge for a predefined period of time.

In addition, a video camera was installed on the tied arch span. The camera was triggered by the logger when a moving vehicle caused the strain to reach a certain value in a particular channel. When the camera was triggered, a video was recorded for a predefined period of time. The use of triggered video permitted the accurate identification of the configuration, position, and number of trucks producing a specific large stress cycle on the Tied Arch. Since Span 25 and 26 were only a few hundred yards north of the tied arch span, the data collected at these spans could also be correlated with the video records.

Stress-range histograms were developed at each location monitored. However, only locations selected based on the results of the controlled load tests and monitoring of random traffic while on site were investigated. Remote communication with the logger and camera was established using a high-speed internet connection. The remote communication allowed program upload and data download to be performed from the ATLSS Research Center in Bethlehem, PA.
1.5 Controlled Load Testing

A series of controlled load tests were conducted using a test truck with three main axles and a fourth floating rear axle. The test was conducted with all four axles carrying the load of the truck (i.e. floating axle was in the ‘down’ position). The test truck was fully loaded with gravel resulting in a gross vehicle weight (GVW) of 72,100 pounds. Figure 1.4 shows the test truck used in testing. Table 1.1 contains the weight at each axle. Table 1.2 provides the dimensions of the test truck.

![Test truck](image)

Figure 1.4 – Test truck used in the controlled load testing

<table>
<thead>
<tr>
<th>Test Description</th>
<th>Rear Axle Type</th>
<th>Front Axle Load (lb)</th>
<th>First Rear axle Load (lb)</th>
<th>Group Rear axle Load (lb)</th>
<th>GVW(^1) (lb)</th>
<th>Date of Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>Controlled Load Tests</td>
<td>Triaxle</td>
<td>20,500</td>
<td>27,380</td>
<td>24,220</td>
<td>72,100</td>
<td>March 18, 2004</td>
</tr>
</tbody>
</table>

Note:  
1. GVW = Gross Vehicle Weight

Table 1.1 – Test truck axle load data
Table 1.2 – Geometry of Test truck used for controlled load tests

The controlled load tests were conducted between the hours of 11 PM and 5 AM on the night of March 17-18, 2004. The Pennsylvania State Police were present to close all lanes of I-79 to traffic during each test. It was noted however that despite closing the interstate, stray vehicles crossed the bridge during some tests after entering the interstate from one of the entrance ramps which were not closed. However, this did not compromise the test results.

The tests consisted of a series of crawl and dynamic tests. For the crawl tests, the test truck was driven across the bridges at approximately 10 miles per hour. The dynamic tests were conducted with the truck traveling at speeds of approximately 35 to 50 miles per hour (The maximum which could be safely attained within the limits of the lane closure).

As discussed previously, data loggers were installed at three locations during the controlled load tests. Each data logger was manned by an ATLSS researcher. In addition, an ATLSS researcher rode with the test truck and recorded lane position, travel speed and other pertinent information. After receiving word from the State Police that the interstate had been closed, the test truck was started from Neville Island. It proceeded northbound across the bridge in either the inside, middle or outside lane (denoted lanes 1, 2, and 3, respectively). It would then exit the interstate, turn around and then proceed southbound in the same lane (1, 2, or 3) and exit back onto Neville Island (using Ramp J). At this point traffic on I-79 was reopened by the state police. This was repeated for each lane. The researchers remained in radio contact with each other the test truck, and the state police for the duration of the testing so the data loggers could be started and stopped at the appropriate times.
1.5.1 Summary of Controlled Load Tests

A summary of the controlled load test data from ramp J, the tied arch, and spans 25-26 are presented in Tables 1.3, 1.4, and 1.5 respectively.

<table>
<thead>
<tr>
<th>Test</th>
<th>Speed</th>
<th>Direction</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>CRWL_LS1.Dat</td>
<td>Crawl</td>
<td>SB</td>
<td>Hug Left Shoulder</td>
</tr>
<tr>
<td>CRWL_CL1.Dat</td>
<td>Crawl</td>
<td>SB</td>
<td>Center line of lane</td>
</tr>
<tr>
<td>CRWL_RS1.Dat</td>
<td>Crawl</td>
<td>SB</td>
<td>Hug Right Shoulder</td>
</tr>
<tr>
<td>CRWL_LS2.Dat</td>
<td>Crawl</td>
<td>SB</td>
<td>Hug Left Shoulder</td>
</tr>
<tr>
<td>CRWL_RS2.Dat</td>
<td>Crawl</td>
<td>SB</td>
<td>Hug right Shoulder</td>
</tr>
<tr>
<td>DYN_1.DAT</td>
<td>Dynamic 30 mph</td>
<td>SB</td>
<td>Center Line of Lane</td>
</tr>
<tr>
<td>DYN_2.DAT</td>
<td>Dynamic 30 mph</td>
<td>SB</td>
<td>Center Line of Lane</td>
</tr>
<tr>
<td>DYN_3.DAT</td>
<td>Dynamic 35 mph</td>
<td>SB</td>
<td>Center Line of Lane</td>
</tr>
</tbody>
</table>

Table 1.3 – Summary of controlled load test data – ramp J

<table>
<thead>
<tr>
<th>Test</th>
<th>Speed</th>
<th>Direction</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>CT_NS_L1.DAT</td>
<td>Crawl</td>
<td>NB/SB</td>
<td>Inside Lane</td>
</tr>
<tr>
<td>CT_NS_L2.DAT</td>
<td>Crawl</td>
<td>NB/SB</td>
<td>Middle Lane</td>
</tr>
<tr>
<td>CT_NS_L3.DAT</td>
<td>Crawl</td>
<td>NB/SB</td>
<td>Outside Lane</td>
</tr>
<tr>
<td>DT_NS_L3.DAT</td>
<td>Dynamic 35-45 mph NB</td>
<td>NB/SB</td>
<td>Outside Lane</td>
</tr>
<tr>
<td>DT_NS_L2.DAT</td>
<td>Dynamic 35-50 mph NB</td>
<td>NB/SB</td>
<td>Middle Lane</td>
</tr>
<tr>
<td>DT_NS_L1.DAT</td>
<td>Dynamic 35-45 mph NB</td>
<td>NB/SB</td>
<td>Inside Lane</td>
</tr>
<tr>
<td>DT_NSL1B.DAT</td>
<td>Dynamic 40-45 mph SB</td>
<td>SB</td>
<td>Inside Lane</td>
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</table>

Table 1.4 – Summary of controlled load test data – tied arch
<table>
<thead>
<tr>
<th>Test</th>
<th>Speed</th>
<th>Direction</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>CRL25L1S</td>
<td>Crawl</td>
<td>SB</td>
<td>Inside Lane</td>
</tr>
<tr>
<td>CR25L2NS.DAT</td>
<td>Crawl</td>
<td>NB/SB</td>
<td>Middle Lane</td>
</tr>
<tr>
<td>CR25L3NS.DAT</td>
<td>Crawl</td>
<td>NB/SB</td>
<td>Outside Lane</td>
</tr>
<tr>
<td>C25L1NS1.DAT</td>
<td>Crawl</td>
<td>NB/SB</td>
<td>Inside Lane</td>
</tr>
<tr>
<td>C25L2NS1.DAT</td>
<td>Crawl</td>
<td>NB/SB</td>
<td>Middle Lane</td>
</tr>
<tr>
<td>C25L3N1.DAT</td>
<td>Crawl</td>
<td>NB</td>
<td>Outside Lane</td>
</tr>
<tr>
<td>CRL25L1S</td>
<td>Crawl</td>
<td>SB</td>
<td>Inside Lane</td>
</tr>
<tr>
<td>D25L3NS1.DAT</td>
<td>Dynamic 45mph NB</td>
<td>NB/SB</td>
<td>Outside Lane</td>
</tr>
<tr>
<td>D25L2NS1.DAT</td>
<td>Dynamic</td>
<td>NB/SB</td>
<td>Middle Lane</td>
</tr>
<tr>
<td>D25L1N1.DAT</td>
<td>Dynamic</td>
<td>NB</td>
<td>Inside Lane</td>
</tr>
<tr>
<td>D25L1S1.DAT</td>
<td>Dynamic</td>
<td>SB</td>
<td>Inside Lane</td>
</tr>
</tbody>
</table>

Table 1.5 – Summary of controlled load test data – spans 25 & 26
2.0  **Ramp J**

2.1  **Bridge Description – Ramp J**

Ramp J carries I-79 south bound traffic to Neville Island. The ramp provides a single driving lane and two side shoulders. The bridge consists of seven spans identified as span J1 to span J7. The instrumented portion of the bridge consists of a two-span continuous unit consisting of spans J1 and J2. The length of the first span, J1, is 124 feet 9 5/8 inches while the length of the second span, J2, is 124 feet 7 5/16 inches on centerline. The bridge deck is detailed as non-composite and supported by two plate girders and transverse floorbeams.

2.2  **Summary of Instrumentation Layout – Ramp J**

The following section summarizes the instrumentation plan for Ramp J. A detailed instrumentation plan is provided in Appendix A.

2.2.1  **Strain Gages on Main Girders**

Gages were located on the sides of the bottom flange of both main girders where knee-braces frame into the girders at spans J1 and J2. Two gages were positioned in the longitudinal direction next to the toe of the fillet welds used for welding the floorbeam tab plates to the main girders. These details are classified as category E details. Figure 2.1 shows a typical detail described and the location of the gage. At each section, gages were also installed on the outside of the girder flange. For example CH_9 was installed directly opposite to CH_11. Since gages were installed on both the inside and outside edges of a bottom flange, the amount of lateral flange bending could also be quantified.
Figure 2.1 – Strain gage installed at weld toe on side of bottom flange of girder G2 in span J2 at floorbeam FB.10 (other locations similar)

In addition to installing gages on the bottom flanges, a single gage was installed on the bottom face of the top flange. The purpose of these gages was to investigate the degree in which the bridge deck and the girders act compositely. Figure 2.2 shows the location of one of the gages (CH_34).

Figure 2.2 – Gage installed on the bottom face of the top flange of girder G1 in span J1 at floorbeam FB.9
Gages were also installed in span J1 on the girder web adjacent to the termination of longitudinal stiffeners at or near the web gap area. These are classified as category E details. A typical installation is shown in Figure 2.3.

Figure 2.3 – Gage (CH_29) installed in span J1 on girder G1 at floorbeam FB.8 at a longitudinal stiffener termination

2.2.2 Strain Gages on Tab Plate

One gage was installed on the connection plate used for attaching the floorbeam knee-brace to the main girder. The gage was located in span J2 on girder G2 at floorbeam FB.5. The gage was installed at the centerline of the connection plate as shown in Figure 2.4.

Figure 2.4 – Strain gage (CH_15) installed in span J2 on girder G2 at the center line of connection plate at floorbeam FB.5
2.2.3 Strain Gages on Floorbeams

Uniaxial strain gages were installed in span J1 on the top and bottom flanges of floorbeam FB.9. Specifically, gages were installed at midspan of the floorbeam and six inches away from the knee-brace connection. Figure 2.5 shows CH_25 and CH_26 installed on floorbeam FB.9 in span J1 six inches away from the knee-brace connection. The gages were used to establish the level of composite action in the floorbeam and gain general insight onto the behavior of the knee-brace connection.

Figure 2.5 – Strain gages (CH_25 and CH_26) installed in span J1 at the floorbeam FB.9 to knee-brace connection
2.3 Results of Controlled Load Tests – Ramp J

The results of the controlled static and dynamic load tests conducted at ramp J are discussed in this section

2.3.1 General Response

Maximum response of any of the instrumented details appeared to be when the test truck was located directly over the detail. Although the bridge was designed and constructed as non-composite (i.e. no shear studs), a review of the data revealed that a high level of composite action exists in the bridge. The composite action was observed between the deck and the primary girders as well as the deck and the floorbeams. Figure 2.6 shows the time history response for CH_19 and CH_22 installed on girder G2 in span J1 at floorbeam FB.9. The gages were located on the outside of the bottom flange and the outside of the top flange of girder G2 respectively. The figure clearly shows that CH_22 installed on the top flange experienced a very low magnitude of stress suggesting that the neutral axis of the cross section is located near the top flange and the bridge is behaving compositely. Similar behavior was observed at all locations where a gage was installed on the top flange.

![Figure 2.6](image)

Figure 2.6 – Response of CH_19 and CH_22 installed in span J1 on the bottom and the top flanges of girder G2 respectively, at floorbeam FB.9, as the test truck passed over the left shoulder in the crawl test (CRWL_LS2)

A significant magnitude of lateral bending moment was observed in girders G1 and G2. As noted in [1], in horizontally curved girders, the lateral bending of the flanges vary dramatically in magnitude and direction along the span with lateral moment peaks occurring at the cross-frame location. Figure 2.7 shows the time history response and the magnitude of the stresses due to both primary and lateral moments in span J1 in girder G1 at floorbeam FB.9.
2.3.2 Repeatability of Data

As indicated in Table 1.3, both the crawl and the dynamic tests were repeated. The data obtained show consistency between repeated tests except in the case when the test truck was passing over the left shoulder in the crawl test. The data shows disagreement in the results between CRWL_LS1.DAT and CRWL_LS2.DAT. A close review of the data indicates that in CRWL_LS1.DAT the truck was not properly positioned over the left shoulder, but rather closer to the center lane. Therefore, the result of the crawl test CRWL_LS1.DAT will be disregarded. It was also observed that the time history response of the installed channels in the crawl test where the test truck was passing over the center lane were almost identical to that of the dynamic test where the test truck was passing over the center lane as well. Hence, the dynamic amplification was not substantial.

2.3.3 Stresses in Girder Flange at Tab Plate Connection

One of the details in which fatigue cracks are of concern are those where tab plates are welded to the side of the girder flange as shown in Figure 2.1 and Figure 2.4. The tab plates were used to transfer part of the load from the knee braces of the floorbeams to the flanges of the girders.

Five details on the bridge similar to the one described above were chosen for the installation of gages (three gages at every detail) to quantify the magnitude of live load stresses. Specifically CH_19, CH_20, and CH_21 (will be referred to as J1G2FB9) were installed in span J1 on girder G2 at floorbeam FB.9; CH_31, CH_32, and CH_33 (J1G1FB9) were installed in span J1 on girder G1 at floorbeam FB.9; CH_12, CH_13, and CH_14 (J2G2FB5) were installed in span J2 on girder G2 at floorbeam FB.5, CH_16, CH_17, and CH_18 (J2G1FB5) were installed in span J2 on girder G1 at floorbeam FB.5,
and finally CH_9, CH_10, and CH_11 (J2G2FB10) were installed in span J2 on girder G2 at floorbeam FB.10.

At every detail three gages were installed in the longitudinal direction. Two gages were installed on the interior side of the girder flange next to the toe of the transverse fillets used for connecting the tab plates to the girder flange. The third gage was installed on the exterior side of the flange (no weld present) by projecting a perpendicular line across the flange from one of the two previously installed gages.

Detail J1G1FB9, J2G2FB5, and J2G1FB5 experienced similar behavior as the test truck passed directly over the detail. CH_31, CH_32, and CH_33 (J1G1FB9) will be discussed in detail since a consistent response of the three details was observed.

The time history response for the three channels located on the bottom flange of girder G1 when the test truck was passing over the right shoulder is shown in Figure 2.8. As the figure shows, CH_31 and CH_32 located next to the transverse fillets experienced high stress ranges, while CH_33 (Located opposite of CH_32) experienced a lower stress range (approximately half). It is believed that compressive stresses resulting from the lateral bending exist on the exterior side of the flange when vertical or in-plane stresses are greatest. The compressive stresses, when superimposed with the tensile primary bending stresses, reduce the overall stresses experienced at the exterior gage.

It is important to note that the magnitude of stresses in all three channels was reduced when the test truck is positioned over the center lane, and was even further reduced when the test truck is over the left shoulder (as expected). Likewise, the other two groups of gages located at similar details experienced almost the same behavior with similar magnitude of stresses. Table 2.1 lists the maximum, the minimum, and the stress range experienced by all 15 channels in all crawl tests.

Figure 2.8 – Response of CH-31, CH_32, and CH_33 installed in span J1 on bottom flange of girder G1 at floorbeam 9 as the test truck crossed over the right shoulder in the crawl test (CRWL_RS2)
Table 2.1 – Maximum stress, minimum stress, and stress-range (ksi) of channels installed on flange of girders G1 and G2 at tab plate connections at floorbeams FB.9, FB.5, and FB.10

Detail J2G2FB10 (CH_9, CH_10, and CH_11) at floorbeam FB.10 experienced significantly lower magnitude of stresses compared to the gages installed at the three locations previously discussed. It is believed that the reason for this was the detail at which CH_9, CH_10, and CH_11 were installed was located close to the pier at 1/4 span.
This resulted in lower bending moment than that experienced by the other three details which were located at 1/3 span.

It is worth mentioning that, as expected, the gages installed on span J1 (eg. CH_31, CH_32, and CH_33) underwent tensile stresses followed by compressive stresses, while the gages installed on span J2 underwent compressive stresses followed by tensile stresses. This is because the spans are continuous where the behavior on one side of the pier should be opposite to that on the other side as the test truck is passing over the lane.

2.3.4 Stress in Tab Plate

To gain some insight respecting the load transfer mechanism between the knee brace and the main girders, one channel (CH_15) was installed at the center line on the bottom face of the tab plate. This plate connects the knee brace to the bottom flange of the main girder (G1) as shown in Figure 2.4.

In all controlled tests performed, the live load stresses measured by CH_15 were approximately zero (Figure 2.9). It is believed that the torsional flexibility of the girder is responsible for the low stresses measured. Low rotation restraint is provided by the main girder at that location. However, similar details could experience higher magnitude of stresses if they are closer to the pier as more rotation restraint could be provided by the pier. Table 2.2 summarizes the maximum stress, the minimum stress and the stress range measured by the channel in the crawl tests.

![Graph showing stress vs. time for CH_15](image)

Figure 2.9 – Response of CH_15 installed in span J2 on girder G2 at floorbeam FB.5 at center line of tap plate as the 72 kip truck crossed over the left shoulder in test (CRW1_CL1)
2.3.5 Stresses in Girder Web at Longitudinal Stiffener Termination

The termination of longitudinal stiffeners are fatigue prone details. Two locations of such details were selected for gage installation. The first location was in span J1 on the web of girder G2 at approximately 3'-3” away from floorbeam FB.8. The second location was in span J1 on the web of girder G1 at floorbeam FB.8. At every detail two strain gages were installed. One gage was installed in the web gap area on the interior face at the termination of the longitudinal stiffener. The other gage was installed on the other side of the web directly opposite the interior gage. The gap is the measure of the distance on the web between the termination of the longitudinal stiffener welded on the web and the toe of the fillet used for welding the transverse stiffener to the web (refer to Appendix A for detailed drawings). The longitudinal stiffener was present on one side only.

The measured web gap at the two locations specified above is listed in Table 2.3 below. Figure 2.10 and Figure 2.11 show the time history response for the four gages. As expected, very low stresses were measured at CH_30 and CH_23. CH_30 and CH_23 were installed on the opposite side of channels CH_29 and CH_24 respectively. A summary of the maximum stress, minimum stress and stress range experienced by all four gages in the crawl tests is listed in Table 2.4.

<table>
<thead>
<tr>
<th>Summary of web gap sizes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
</tr>
<tr>
<td>Web gap size</td>
</tr>
</tbody>
</table>

Table 2.3 – Summary of web gap sizes at the two instrumented locations

Table 2.2 – Maximum stress, minimum stress, and stress-range (ksi) of CH_15 installed in span J2 on girder G2 at floorbeam FB.5 at the center line on the bottom face of the tab plate.

<table>
<thead>
<tr>
<th>Truck in lane</th>
<th>Tab plate (G2, CH_15)</th>
<th>( \sigma_{\text{max}} )</th>
<th>( \sigma_{\text{min}} )</th>
<th>( Sr )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Right shoulder</td>
<td>0.1</td>
<td>0.0</td>
<td>0.1</td>
<td></td>
</tr>
<tr>
<td>Center lane</td>
<td>0.1</td>
<td>-0.2</td>
<td>0.3</td>
<td></td>
</tr>
<tr>
<td>Left shoulder</td>
<td>0.1</td>
<td>-0.1</td>
<td>0.2</td>
<td></td>
</tr>
</tbody>
</table>
Figure 2.10 – Response of CH_29 and CH_30 located in span J1 on web of girder G1 at floorbeam FB.8 as the test truck crossed over the right shoulder in the crawl test (CRWL_RS1)

Figure 2.11 – Response of CH_23 and CH_24 located in span J1 on web of girder G2 at approximately 3’-3’ away from floorbeam FB.8 as the test truck crossed over the left shoulder in the crawl test (CRWL_RS1)
Table 2.4 – Maximum stress, minimum stress and stress-range (ksi) at longitudinal stiffener web gap details

### 2.3.6 Stresses in Top Flanges of Main Girders

Two gages were installed on the top flanges directly above the gages installed on the exterior edge of the bottom flanges. Specifically, CH_22 was installed directly on girder G2 directly above CH_19 and CH_34 was installed on girder G1 directly above CH_33. Because the top flange is partially embedded in the concrete deck, the gages were installed on the bottom face of the top girder flange, instead of on the edge of the flange as was done on the bottom flange. The main purpose of installing gages on the top flange is to investigate the degree in which the top flanges and the deck are acting compositely.

Figure 2.6 shows the time history response of CH_19 installed on the bottom flange in span J1 on girder G2 at floorbeam FB.9 and CH_22 installed on the top flange directly above CH_19 in span J1 on Girder G2 at floorbeam FB.9. The responses shown indicate high magnitude of stresses for CH_19 and very low magnitude of stresses for CH_22. This suggests that the neutral axis is located near the top flange and that the deck is acting compositely. If composite action did not exist, then the neutral axis would have been shifted down, resulting in higher magnitude of stresses at the top flange. Similar behavior was observed regardless of the transverse position of the test truck. Table 2.5 lists the maximum stress, minimum stress and stress range experienced by the top gages installed on the top flanges in the crawl tests.
Table 2.4 – Maximum stress, minimum stress and stress-range (ksi) for CH_19, CH_22, CH_33, and CH_34 installed on top and bottom flange of girder G1 and G2 in span J1 at floorbeam FB.9

<table>
<thead>
<tr>
<th>Truck in lane</th>
<th>Bottom flange (G1, CH_34)</th>
<th>Top flange (G2, CH_22)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\sigma_{\text{max}}$</td>
<td>$\sigma_{\text{min}}$</td>
</tr>
<tr>
<td>Right shoulder</td>
<td>0.2</td>
<td>0.0</td>
</tr>
<tr>
<td>Center lane</td>
<td>0.0</td>
<td>-0.1</td>
</tr>
<tr>
<td>Left shoulder</td>
<td>0.0</td>
<td>-0.1</td>
</tr>
</tbody>
</table>

2.3.7 Stresses in Floorbeams

One floorbeam was chosen for instrumentation. Four channels were installed on the floorbeam at two different sections. Gages were installed 6 inches away from the knee-brace-to-floorbeam connection and at midspan of the floorbeam. As shown in Figure 2.5, two channels were installed (i.e. one on the bottom face of the bottom flange and one on the bottom face of the top flange) at each cross section. Figure 2.13 shows the response of the both the top and the bottom channels located 6 inches away from the knee-brace-to-floorbeam connection, while Figure 2.14 shows the response of the channels located at the midspan of the floorbeam for the same test. During this test, the test truck was located in the center lane and was traveling with a speed of 10 mph.

It can be seen that the stresses at midspan of the floorbeam (Figure 2.14) are significantly higher than the stresses near the floorbeam-to-knee-brace connection (Figure 2.13). This suggests that the floorbeam is behaving as a simply supported beam in which the maximum response is located at midspan of the beam. Furthermore, the data demonstrate that the floorbeam acts compositely with the concrete deck since the magnitude of stresses at the top flanges is much lower than the bottom flange stresses. This behavior was consistent regardless of the transverse position of the truck as shown in Table 2.5.
Figure 2.13 – Response at top and bottom flanges of floorbeam FB.9 at 6” away from knee brace in span J1 as the test truck crossed over the center lane in the crawl test (CRWL_CL1)

Figure 2.14 – Response of CH_27 and CH_28 installed in span J1 at midspan of floorbeam FB.9 as the test truck crossed over the center lane in the crawl test (CRWL_CL1)
### Floorbeam 9, 6” away from knee-brace

<table>
<thead>
<tr>
<th>Truck in lane</th>
<th>Top flange (FB. 9, CH_25)</th>
<th>Bottom flange (FB. 9, CH_26)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\sigma_{\text{max}}$</td>
<td>$\sigma_{\text{min}}$</td>
</tr>
<tr>
<td>Right shoulder</td>
<td>0.0 0.0 0.0</td>
<td>0.1 -0.1 0.2</td>
</tr>
<tr>
<td>Center lane</td>
<td>0.1 0.0 0.1</td>
<td>0.8 0.0 0.8</td>
</tr>
<tr>
<td>Left shoulder</td>
<td>0.3 0.0 0.3</td>
<td>0.7 -0.1 0.8</td>
</tr>
</tbody>
</table>

### Midspan of the floorbeam 9

<table>
<thead>
<tr>
<th>Truck in lane</th>
<th>Top flange (FB. 9, CH_27)</th>
<th>Bottom flange (FB. 9, CH_28)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\sigma_{\text{max}}$</td>
<td>$\sigma_{\text{min}}$</td>
</tr>
<tr>
<td>Right shoulder</td>
<td>0.1 0.0 0.1</td>
<td>0.6 -0.1 0.7</td>
</tr>
<tr>
<td>Center lane</td>
<td>0.4 0.0 0.4</td>
<td>2.4 0.0 2.4</td>
</tr>
<tr>
<td>Left shoulder</td>
<td>0.0 0.0 0.0</td>
<td>0.4 0.0 0.4</td>
</tr>
</tbody>
</table>

Table 2.5 – Maximum stress, minimum stress and stress-range (ksi) for CH_25 through CH_28 installed in span J1 on floorbeam FB.9
2.4 Long-term Monitoring

The Long-term monitoring of ramp J was conducted from March 27, 2004 to April 26, 2004 for a total of 29 days. All twenty-six gages installed on the bridge were chosen for the long term monitoring. Recording of the data in all channels was triggered when a predefined stress value of 4.5 ksi or more was measured in CH_31. CH_31 was selected as the “trigger” channel as it consistently responded to passing trucks. For every trigger event, five seconds of data prior to the event and five seconds after the event were recorded.

Additionally, for the entire monitoring period, stress-range histograms for each channel were generated by the data logger using the rainflow cycle counting technique.

2.4.1 Results of Long-term Monitoring

An estimate of the magnitude of stresses caused by the normal daily traffic could be established by reviewing the data collected during the monitoring period. Stresses of higher magnitude than produced by the test truck were observed. Such an observation is not uncommon and is typically the result of multiple vehicles crossing the bridge. However, it is unlikely that the high stress values measured on ramp J were the result of multiple vehicles passing over the bridge since it consists of only one lane. Hence, the larger stress-range cycles were likely the result of trucks with gross or axle weight heavier than the 72.1 kip test truck. An in-depth discussion of the methodology used for the fatigue evaluation can be found in Appendix B.

2.4.2 Stresses in Girder Flange at Tab Plate Connection

Stress-range cycles higher than the constant amplitude fatigue limit (CAFL) were measured in the bottom flange at the tab plate details where the plates are welded to the bottom flange of the main curved girders. Figure 2.15 presents the stress-range histogram for channels CH_31 and CH_32. As can be seen in the inset of Figure 2.15, stress range cycles up to 8.5 ksi in both channels were recorded. The detail where the channels were installed is classified as a category E detail per AASHTO specifications. A stress range truncation level of 1 ksi was selected for producing the histogram. Typically a truncation level of 1/4 to 1/3 times the CAFL of the detail is selected. Both channels experienced a considerable number of cycles above the CAFL of the detail (4.5 ksi). A review of the stress-range histogram indicates that during the one month monitoring period, 1374 cycles exceeded the CAFL of 4.5 ksi in CH_31, while 1034 cycles exceeded the CAFL of 4.5 ksi in CH_32.

A summary of the magnitude of the maximum stress range, effective stress range, number of cycles measured per day, and the remaining life for the details is presented in Table 2.6. As can be seen in Table 2.6, the minimum calculated remaining fatigue life for all locations is over 100 years. It must be noted that concluding that a detail would have an infinite life was based on the fact that the effective stress range was below the CAFL for the detail and that the frequency of cycles exceeding the CAFL was less than 1/10,000. The total life for details with S_{eff} exceeding the CAFL was calculated directly using the appropriate S–N curve. The remaining life was calculated by taking the total life, and subtracting from it the number of cycles experienced by the detail after being in service for 30 years. Thus the estimate of 100 years is from 2004 with the assumption that traffic patterns will stay the same. This approach also conservatively assumes that the 2004 traffic volumes were representative of the past 30 years.
Figure 2.15 – Stress range histogram for CH_31 and CH_32

<table>
<thead>
<tr>
<th>Channel</th>
<th>S&lt;sub&gt;max&lt;/sub&gt; ksi</th>
<th>Cycles &gt; CAFL</th>
<th>S&lt;sub&gt;eff&lt;/sub&gt; ksi</th>
<th>Cycles / Day</th>
<th>Remaining Life (Years)</th>
<th>Category</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_10</td>
<td>3.3</td>
<td>0</td>
<td>1.6</td>
<td>145</td>
<td>Infinite</td>
<td>E</td>
<td>G2, FB. 10</td>
</tr>
<tr>
<td>CH_11</td>
<td>2.8</td>
<td>0</td>
<td>1.2</td>
<td>126</td>
<td>Infinite</td>
<td>E</td>
<td>G2, FB. 10</td>
</tr>
<tr>
<td>CH_13</td>
<td>3.8</td>
<td>0</td>
<td>1.8</td>
<td>257</td>
<td>Infinite</td>
<td>E</td>
<td>G2, FB. 5</td>
</tr>
<tr>
<td>CH_14</td>
<td>5.3</td>
<td>6</td>
<td>0.2</td>
<td>464</td>
<td>Infinite</td>
<td>E</td>
<td>G2, FB. 5</td>
</tr>
<tr>
<td>CH_16</td>
<td>7.3</td>
<td>758</td>
<td>3.4</td>
<td>735</td>
<td>Over 100</td>
<td>E</td>
<td>G1, FB. 5</td>
</tr>
<tr>
<td>CH_17</td>
<td>5.8</td>
<td>185</td>
<td>1.1</td>
<td>570</td>
<td>Over 100</td>
<td>E</td>
<td>G1, FB. 5</td>
</tr>
<tr>
<td>CH_20</td>
<td>6.3</td>
<td>121</td>
<td>0.7</td>
<td>540</td>
<td>Over 100</td>
<td>E</td>
<td>G2, FB. 9</td>
</tr>
<tr>
<td>CH_21</td>
<td>7.3</td>
<td>493</td>
<td>2.6</td>
<td>642</td>
<td>Over 100</td>
<td>E</td>
<td>G2, FB. 9</td>
</tr>
<tr>
<td>CH_31</td>
<td>8.3</td>
<td>1374</td>
<td>5.6</td>
<td>824</td>
<td>Over 100</td>
<td>E</td>
<td>G1, FB. 9</td>
</tr>
<tr>
<td>CH_32</td>
<td>7.8</td>
<td>1034</td>
<td>4.8</td>
<td>724</td>
<td>Over 100</td>
<td>E</td>
<td>G1, FB. 9</td>
</tr>
</tbody>
</table>

Note
1. The effective stress range and cycles per day calculations ignore cycles less than 1.0 ksi

Table 2.6 – Summary of maximum and effective stress ranges on bottom flange adjacent to tab plate
2.4.3 Stresses in Girder Web at Longitudinal Stiffener Termination

Figure 2.16 presents the stress-range histogram for CH_24 and CH_29, which were installed at the termination of longitudinal stiffeners in web gap areas. The figure shows that CH_24 and CH_29 experienced stress ranges higher than the CAFL of the detail, which is classified as category E. As shown in Table 2.7, CH_23 and CH_30, however, experienced very low stress ranges; considerably below the CAFL of the detail. It should be noted that CH_23 and CH_30 are classified as category A details since there is no web attachment on the base metal. Table 2.7 summarizes the results for all four gages.

![Stress-range histogram for CH_24, and CH_29](image)

**Figure 2.16 – Stress range histogram for CH_24, and CH_29**

<table>
<thead>
<tr>
<th>Channel</th>
<th>$S_{\text{max}}$ ksi</th>
<th>Cycles &gt; CAFL</th>
<th>$S_{\text{ref}}$ ksi</th>
<th>Cycles / Day</th>
<th>Remaining Life (Years)</th>
<th>Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_24</td>
<td>7.3</td>
<td>221</td>
<td>1.3</td>
<td>2.3</td>
<td>582</td>
<td>Over 100</td>
</tr>
<tr>
<td>CH_29</td>
<td>5.8</td>
<td>830</td>
<td>0.5</td>
<td>2.1</td>
<td>525</td>
<td>Over 100</td>
</tr>
<tr>
<td>CH_23</td>
<td>1.3</td>
<td>0</td>
<td>0</td>
<td>-</td>
<td>-</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_30</td>
<td>0.8</td>
<td>0</td>
<td>0</td>
<td>-</td>
<td>-</td>
<td>Infinite</td>
</tr>
</tbody>
</table>

**Table 2.7 – Summary of results for CH_24, CH_29, CH_23, and CH_30**

**Note**
1. The effective stress range and cycles per day calculations ignore cycles less than 1.0 ksi
3.0  **Ramp H**

3.1  **Bridge Description – Ramp H**

Ramp H carries traffic from Neville Island onto I-79 south. It is a single lane bridge with two shoulders. The bridge is supported by two curved steel plate girders identified as G1 and G2 spanning over seven portions identified as H1 to H7. The instrumented portion of the bridge consisted of a single span identified as span H6. The length of the span is 65 feet 0 inches. The bridge deck is designed as non-composite with the two primary curved girders as well as the floorbeams.

3.2  **Summary of Instrumentation Layout – Ramp H**

The following section describes the instrumentation plan used for the long-term monitoring. Data were not recorded during the controlled load tests. All measurements were made during the long-term monitoring of random traffic. Details on the location of the strain gages instrumented on the bridges can be found in Appendix A.

3.2.1  **Strain Gages on Girder Web**

Out-of-plane distortion cracking has been known to be the cause of many fatigue cracks that appear in bridges in the US. This type of fatigue cracking has been observed at some locations on Ramp H in the past and was retrofitted.

To assure that the remaining details on the bridge similar to those that cracked previously are not vulnerable to the same type of cracking, strain gages were installed on the web of girder G1 adjacent to the vertical fillet weld attaching the knee-brace connection plate to the girder web at the bottom flange.

Two gages, CH_36 and CH_37, were installed in span H6 on the web of G1 at floorbeam FB.54. CH-36 was located on the inside of the web, while CH_37 was installed on the outside, by perpendicularly projecting CH_36 (See Figure 3.1 and Figure 3.2). Another strain gage CH_35, was installed in span H6 on the web of G1 at the transverse connection plate detail (on the inside) at floorbeam FB.53 and positioned similar to CH_36.
3.2.2 Displacement Sensors

To correlate between the stress measured at CH_36 and the potential out-of-plane distortions, an LVDT was placed in span H6 to measure the relative displacement between the transverse connection plate (knee-brace) and the flange of girder G1 at floorbeam FB.52 as shown in Figure 3.3.
3.3 Long-term Monitoring

The long-term monitoring of ramp H was conducted from March 27, 2004 to April 26, 2004 for a total of 29.34 days. All four channels installed on the bridge were included in the long term monitoring program. Recording of the data in all channels was triggered when a predefined stress value of 1ksi or more was measured in CH_35. Stress-range histograms for each channel were generated by the data logger using the rainflow cycle counting method.

3.3.1 Results of Long-term Monitoring

An estimate of the magnitude of stresses caused by the normal daily traffic could be established by reviewing the data collected during the monitoring period. The result of the long-term monitoring (as will be discussed below) suggests that no fatigue cracking due to out-of-plane distortion is likely at the instrumented details.

3.3.2 Stresses in Girder Web Adjacent to Transverse Knee-Brace

The detail at which the strain gages were installed on the inside of the girder next to the vertical fillet weld attaching the transverse knee-brace to the girder web is classified as category C. A review of the data indicates that both CH_35 and CH_36 (installed on the inside face of the girder) experienced low stress range cycles below the CAFL of the details. Figure 3.4 presents the stress-range histogram for CH_36. As can be seen in the figure, no high stress-range cycles were observed. A lower-bound stress range truncation level of 2.5ksi was selected for producing the histogram. Typically the truncation level is 1/4 to 1/3 times the CAFL of the detail. None of the channels experienced any cycles above the CAFL of the detail (10 ksi).

A summary of the magnitude of the stress range, effective stress range, number of cycles measured per day, and the remaining life for the details is presented in Table 3.1.
**Figure 3.4 - Stress-range histogram for CH_36**

**Fatigue Life Calculation Summary**

<table>
<thead>
<tr>
<th>Channel</th>
<th>$S_{\text{max}}$ ksi</th>
<th>Cycles&gt;CAFL</th>
<th>$S_{\text{eff}}$ ksi</th>
<th>Cycles / Day</th>
<th>Remaining Life (Years)</th>
<th>Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_35</td>
<td>3.3</td>
<td>0</td>
<td>0</td>
<td>3.3</td>
<td>2</td>
<td>Infinite</td>
</tr>
<tr>
<td>CH_36</td>
<td>4.8</td>
<td>0</td>
<td>0</td>
<td>3.6</td>
<td>315</td>
<td>Infinite</td>
</tr>
<tr>
<td>CH_37</td>
<td>3.3</td>
<td>0</td>
<td>0</td>
<td>3.25</td>
<td>3</td>
<td>Infinite</td>
</tr>
</tbody>
</table>

**Note**
1. The effective stress range and cycles per day calculations ignore cycles less than 2.5 ksi

**Table 3.1 - Summary of stress-range histogram**
4.0 Spans 25 and 26

4.1 Bridge Description - Spans 25 and 26

These two spans are part of the northern approach spans to the tied arch. These approach spans consist of nearly separate northbound and southbound superstructures. Each side consists of two or three welded plate girders, welded plate girder floorbeams, rolled steel stringers, and an 8 1/2 inch concrete deck separated by a longitudinal joint. Selected floorbeams are made continuous between the northbound and southbound structures with a steel diaphragm with the same cross-sectional dimensions as the floorbeam.

Spans 22 through 25 compose a four span continuous unit adjacent to the tied arch span. Each span is 125 feet. Span 25 is an end span. Spans 26 and 27 are a two span continuous unit. The spans are 157 feet and 163 feet, respectively. Spans 26 and 27 have chevron lateral wind bracing throughout. There is no wind bracing in spans 22 through 25.

4.2 Summary of Instrumentation Layout – Span 25 and 26

The following section summarizes the instrumentation plan. A detailed instrumentation plan is included in Appendix A.

4.2.1 Strain Gages - Span 25

4.2.1.1 Strain Gages on Flange of Main Girders

The north and south bound portions of the bridge are connected by diaphragms spanning 8 feet 6 inches between girders G2 and G3. To understand the global behavior of the bridge and the load distribution between the two sides of the bridge, gages were installed on the bottom flanges of all four girders (G1, G2, G3, and G4) at floorbeams FB.40 (northbound side) and FB.19 (southbound side), at the 2/5 point of span 25. Figure 4.1 shows a typical gage (CH_14) installed on the bottom flange of girder G3.
4.2.1.2 Web Gap at Transverse Stiffener

Out-of-plane distortion cracking is known to occur in bridges when relative displacement takes place between transverse stiffeners and girders webs. The relative displacement is caused by out-of-plane bending of the girder web. Details susceptible to out-of-plane distortion cracking exist at many locations on all of the mainline plate girder spans. As shown in Figure 4.2, gages were installed in the vertical direction on the web of the girder next to the toe of the fillet used for attaching the transverse stiffener to the girder web (CH_11 in the figure), to quantify the stress range at the locations where out-of-plane distortion is expected. It should be noted that out-of-plane distortion cracking is not expected at CH_11. This is because the transverse stiffener is welded to the flange, as shown in Figure 4.2. This strain gage (CH_11) was installed as part of the investigation into the observed “type K” cracking, as noted in the SAI inspection report.

Figure 4.2 - Strain gages CH_10 and CH_11 installed vertically on the transverse stiffener and vertically on web of girder G3 at FB.40
4.2.1.3 Transverse Stiffener Flange Connection

Cracks identified as “type K” cracking, per the SAI routine inspection report, exist at many transverse stiffener-to-girder-flange welded connections throughout all mainline plate girder spans. To quantify stresses at these details, back-to-back gages were installed on the transverse stiffener at the upper weld toe (CH_10, Figure 4.2). Various locations at girders G3 and G4 were instrumented.

4.2.1.4 Diaphragm Type D2 - Structural T Sections

As discussed above, at certain locations, continuity is provided between the northbound and southbound bridges by diaphragms spanning between the interior girders, G2 and G3. Some of these diaphragms are designated as type D2 diaphragms on the design drawings. Full moment connections were provided between this type of diaphragm and the girders G2 and G3.

Structural T sections were used for moment transfer at the diaphragm-to-girder connections. Four T sections at each end of the diaphragm were bolted to the insides of the top and bottom flanges. To quantify the moment transferred by the T sections and the resulting stresses, gages were installed in the transverse direction on the web of the T’s as shown in Figure 4.3.

![Strain gage installed on the web of a T section at a typical type D2 diaphragm](image)

Figure 4.3 - Strain gage installed on the web of a T section at a typical type D2 diaphragm
4.2.1.5 Diaphragm Type D1 – Simple Connections

Strain gages were installed on the flange of a type D1 diaphragm. Although a simple shear connection is used between diaphragm D1 and the main girders G2 and G3, some moment may be transferred through the connection. The beam has a very low span-to-depth ratio of 1.5. Strain gages were installed on the top and bottom flanges of the diaphragm at a distance of 24 inches away from the face of the web of girder G3.

4.2.1.6 Floorbeams

Strain gages were also installed at mid-span at the centerline of the top and bottom flanges of floorbeam FB.40 and FB.41. Floorbeam FB.41 is shown in Figure 4.4. These floorbeams span between girders G3 and G4. Note these gages were installed after the controlled load tests and were operational only during the part of the long-term monitoring. These gages were added to aid in the investigation into the cause of the observed “K” cracking.

Figure 4.4 - Strain gages installed on top and bottom flanges of floorbeam FB.41
4.2.1.7 Stringers

Strain gages were installed on one W24x68 stringer (first stringer east of G3) at two locations centered above floorbeams FB.40 and FB.41. At each location, one gage was mounted to each side of the stringer web (back-to-back) in a vertical orientation, adjacent to the fillet at the bottom flange. These gages measured any out-of-plane bending in the stringer web which might result from relative displacement between the bridge deck and the floorbeams. Figure 4.5a shows a strain gage installed on the web of a stringer. These gages were added to aid in the investigation into the cause of the observed “K” cracking.

Two strain gages were also installed back-to-back on the web of floorbeam FB39 at the top flange cope. Figure 4.5b contains a photograph of these strain gages.

Figure 4.5a - Strain gage installed on the web of a stringer
4.2.2 Strain Gages - Span 26

4.2.2.1 Longitudinal Stiffener Terminations

Four longitudinally oriented strain gages were installed on the web of girders G1, G2, and G3 to measure stresses in the web at the termination of the longitudinal stiffeners on span 26. The longitudinal and transverse stiffeners are present on one side of the girder web only. Typically, gages would be placed in the web gap area (i.e., on the web in the gap between the longitudinal stiffener and the transverse stiffener). Because of the small gap in all locations instrumented, the gages were installed on the web on the other side of the transverse stiffener, as shown in Figure 4.6. Strain gages CH_25 and CH_28 were installed on girders G1 and G3, respectively (CH_28 is shown in Figure 2.6). Strain gages CH_26 and CH_27 were installed on opposite sides of the web (back-to-back) of girder G2.
4.2.2.2 Displacement Sensors - Span 26

To measure the relative out-of-plane displacement between the top flange of girder G3 and the transverse stiffener, three linear variable differential transformers (LVDT) were installed. Figure 4.7 shows a typical installation.
4.3 **Results of Controlled Load Tests**

The results of the controlled static and dynamic load tests are discussed in this section.

4.3.1 **General Response**

Span 25 is the end span of a four-span continuous plate girder bridge. The global behavior of these spans as measured by the four gages on the bottom flanges of the main girders is as expected from simple structural analysis. One of the unique features of this bridge is the periodic connection between floorbeams on north and south bound sides of the bridge. The data indicate that there is noticeable distribution of load across the longitudinal joint separating the north and south bound sides.

More interestingly is the local behavior observed at various details. Due to incompatibility of the floorbeam and the concrete deck, horizontal thrusts are generated at each end of the floorbeam which caused secondary stresses at some details. This will be discussed further below.
4.3.2 Main Girders

As previously mentioned, strain gages oriented in the longitudinal direction were installed on the bottom flanges of all four girders (at FB.40 for G3 and G4 and FB.19 for G1 and G2). As expected, the peak stress was observed when the test truck was passing directly over the instrumented cross-section. Furthermore, there was noticeable distribution of load between girders across the longitudinal joint separating the north and south bound sides of the bridge.

As the test truck runs moved transversely across the bridge, the girders nearest the test truck received the highest stresses. Figure 4.8 shows the time history response for the four strain gages during a crawl test when the test truck was traveling north in the middle lane. Figure 4.9 contains a similar plot for the truck traveling south in the middle lane. It can be seen the behavior is nearly symmetric. When the truck is traveling north (Figure 4.8), the peak stresses occur in the interior girder G3. High stresses are also recorded in girders G2 and G4. However, in girder G1, furthest from the test truck, minimal stresses were measured. The opposite is true for the truck traveling south bound (Figure 4.9). It can be seen that the stresses in girders G1 and G3 have a very similar magnitude of stress. This is due to the sharing of load across the longitudinal joint.

The peak stresses measured in the girder flange gages during the crawl tests are summarized in Table 4.1. The maximum stress range in girders G2 and G3 (= 1.4 ksi) is less than that for girders G1 and G4 (= 2.0 ksi). This is a result of the load sharing between girders G2 and G3.

![Figure 4.8 – Response of bottom flanges of girders G1, G2, G3, and G4 with the test truck traveling north in the middle lane](image)
Figure 4.9 – Response of bottom flanges of girders G1, G2, G3, and G4 with the test truck traveling south in the middle lane

<table>
<thead>
<tr>
<th>Truck in Lane</th>
<th>CH_3 (G1)</th>
<th>CH_13 (G2)</th>
<th>CH_14 (G3)</th>
<th>CH_4 (G4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1N</td>
<td>0.2</td>
<td>1.2</td>
<td>1.4</td>
<td>0.5</td>
</tr>
<tr>
<td>1S</td>
<td>0.7</td>
<td>1.4</td>
<td>1.3</td>
<td>0.4</td>
</tr>
<tr>
<td>2N</td>
<td>0.3</td>
<td>1.0</td>
<td>1.4</td>
<td>0.9</td>
</tr>
<tr>
<td>2S</td>
<td>1.2</td>
<td>1.4</td>
<td>1.0</td>
<td>0.5</td>
</tr>
<tr>
<td>3N</td>
<td>0.3</td>
<td>0.5</td>
<td>0.7</td>
<td>2.0</td>
</tr>
<tr>
<td>3S</td>
<td>1.9</td>
<td>1.0</td>
<td>0.6</td>
<td>0.3</td>
</tr>
</tbody>
</table>

SR_{max} = 1.9  1.4  1.4  2.0

Table 4.1 – Summary of maximum, minimum, and stress ranges (ksi) for strain gages on the bottom flange of the four main girders for various test truck lane positions

Note: Lane 1 is the inside lane, lane 3 is the outside lane
4.3.3 Secondary Stresses – Floorbeams

4.3.3.1 Discussion

As described above, the main girders behave as expected for a four-span continuous bridge. However, when the data from strain gages mounted to the transverse stiffener at the connection to the top flange were analyzed, an unexpected behavior was observed. Gages were mounted vertically back-to-back at the edges of the transverse stiffener-to-flange weld (see Figure 4.2). High stresses were measured at these gages. Furthermore, the magnitude of the stresses was similar indicating that either the entire web-stiffener cruciform was subjected to either uniform axial stress, bending about the web, or a combination of both. Because gages were originally installed on one side of the web only at a particular connection, the components of the stress could not be evaluated (i.e., axial or bending).

Based on the measurements from instrumentation initially installed, it was inferred that the cause of the observed “K” cracking was the result of an incompatibility in floorbeam and deck slab bending, as shown schematically in Figure 4.10. To confirm this, 16 gages were added (subsequent to the controlled load testing) to the north bound side of span 25 at floorbeams FB.40 and FB.41, which would be monitored during the long-term monitoring period. Results of the long-term monitoring will be discussed in detail in the next section. However, for the purposes of describing the behavior which caused these secondary stresses, some results from the long-term monitoring will be presented herein.

A schematic drawing of a common two girder system, with a floorbeam, stringers, and a concrete deck is shown in Figure 4.10. It can be seen that similar to span 25, the deck is supported directly by the girder flanges, and the stringers. It is not resting atop the floorbeam. When load is applied to the deck as shown, the deformation of the deck and floorbeam is compatible with the top flanges of the girders and stringers, but not with the floorbeam. As a result, there is a differential displacement between the deck and floorbeam which is zero at the midspan, and increases to a maximum at the ends. This displacement puts the webs of the stringers and girder into reverse curvature, as shown.

![Figure 4.10 – Schematic drawing showing secondary deformations resulting from the incompatibility between the floorbeam and concrete deck](image-url)
A detail drawing of the girder from span 25 is shown in Figure 4.11. Locations of the strain gages on the top of the transverse stiffener are shown. The displaced shape shown results from a truck over the floorbeam on the left side of the figure. It can be seen that such loading causes tension in the transverse stiffener on the opposite side of the girder web, and compression on the near-side stiffener, shown in red and blue, respectively. It is these stresses that are believed to be the cause of the “type K” cracking observed on the bridge.

Figure 4.11 – Detail showing secondary deformations resulting from the incompatibility between the floorbeam and concrete deck
This phenomena was clearly evident in the long-term data. As discussed, data were collected for a predefined period of time when the stress in a pre-selected strain gage (on the tie girder flange) exceeded a strain threshold, indicating the presence of a heavy vehicle. Three sets of triggers were used: (1) girder G4 indicating passage of a northbound vehicle in the outside lane, (2) girder G3, indicating passage of a north or south bound vehicle in the inside lanes, and (3) girder G1, indicating passage of a southbound vehicle in the outside lane. Figure 4.12 contains a partial strain gage plan for floorbeam FB.41. This floorbeam is the first floorbeam south of Pier 26. Strain gages were placed on the first stringer east of girder G3.

![Figure 4.12 – Partial Strain Gage Plan for floorbeam FB.41](view looking south)

Plotted in Figure 4.13 is a stress time-history for the floorbeam and stringer gages during the passage of a random heavy vehicle traveling north in the outside lane. When the truck was in the outside north bound lane, the floorbeam was put into positive bending, since there was negative stress at the top (blue line) and positive stress at the bottom (red line). The stringer web was subjected to comparable bending stresses, due to the incompatibility of the deck and floorbeam, as depicted in Figure 4.10. Note that the side closest to the load was put into tension (magenta line) while the other side was in compression (green line).

Figure 4.14 contains a time-history plot of stresses in the transverse stiffeners. It can be seen that for any pair of back-to-back gages, the stresses in the pair are very similar. This indicates that there is little out-of-plane bending of the transverse stiffener plates. However, considering the gages on the stiffener at either side of girder G3, it can be seen that the stresses were opposite in sign, indicating that the stiffener was bending about the girder web. This bending was due to the incompatibility of the concrete deck and the floorbeam (see Figures 4.10 and 4.11). It can be seen that high stresses (up to 17 ksi) were induced in the transverse stiffener.

Figures 4.15 and 4.16 contain the same data as Figures 4.13 and 4.14, for a random heavy vehicle traveling south in the outside lane. The presence of the truck in the south bound lanes causes the north bound side of the bridge to “uplift”. The incompatibility of the deck and floorbeam still exists, however, the signs of the stresses produced are opposite from those caused by a north bound truck. This can be seen by comparing Figure 4.13 to 4.15 and Figure 4.14 to 4.16.
Figure 4.13 – Time-history plot of measured floorbeam and stringer stresses for passage of a random heavy vehicle traveling **north** across span 25 in the outside lane (near G4)

Figure 4.14 – Time-history plot of measured stresses in the transverse stiffeners for passage of a random heavy vehicle traveling **north** across span 25 in the outside lane (near G4)
Figure 4.15 – Time-history plot of measured floorbeam and stringer stresses for passage of a random heavy vehicle traveling **south** across span 25 in the outside lane (near G1)

Figure 4.16 – Time-history plot of measured stresses in the transverse stiffeners for passage of a random heavy vehicle traveling **south** across span 25 in the outside lane (near G1)
It should be noted that the stress magnitudes for each of these random truck positions cannot be compared since the weight of the vehicles is unknown. The plots are presented solely to describe the behavior of the bridge.

**4.3.3.2 Transverse Stiffener/Flange Connection**

Data collected at the strain gages mounted to the transverse stiffeners at the toe of the weld to the top flange of the girder during controlled load testing are presented in this section. These gages were installed similarly to those discussed above in Section 4.3.3.1. The response measured at the gages presently discussed are consistent with that discussed above and further confirm the incompatibility issue.

Figure 4.17 contains a stress time-history plot for strain gages mounted to a transverse stiffener at two locations. Two gages are installed at each location. The stress peaks when the test truck is directly over the connection. Since CH_9 and CH_10 are located on the diaphragm side of girder G3, they are subjected to tension stress with the test truck in the north bound lanes. Strain gages CH_15 and CH_16 are on the floorbeam side of girder G3 and therefore are put into compression with the passing of a northbound test truck.

A summary of the maximum, minimum and stress ranges for each channel, for each test is shown in Table 4.2. It can be seen that the highest stress ranges are produced when the test truck is in the middle north bound lane. Furthermore, the stress ranges generally decrease as the test truck position moves from east to west.

Note that strain gages CH_15 and CH_16 are located at a floorbeam with no diaphragm, whereas CH_9 and 10 are at a location with a diaphragm. At CH_15 and CH_16, the stresses in the transverse stiffener are much lower when the truck is in the south bound lanes. For example, at CH_15 the stress is -7.4 ksi with the truck in the northbound inside lane, while it is only 0.7 ksi with the truck in the inside southbound lane.

At CH_9 and 10, there is significant stress transferred across the diaphragm. For example, at CH_9 the stress is 5.0 ksi with the truck in the northbound inside lane, and -2.3 ksi with the truck in the southbound inside lane.
Figure 4.17 – Time-history plot of measured stresses in transverse stiffeners during controlled load tests with test truck traveling \textbf{north} across span 25 in the outside lane

Table 4.2 – Summary of maximum, minimum, and stress ranges (in ksi) for strain gages on the transverse stiffeners at the stiffener/flange weld for various test truck lane positions

Note: \textit{Lane 1 is the inside lane, lane 3 is the outside lane}
4.3.4 Diaphragms

As described above, two types of diaphragms are present on the bridge connecting girders G2 and G3. The web depth and flange width of the diaphragms match the floorbeams on either side (the floorbeams have varying flange thicknesses). The difference between the two diaphragm types lies in the end connections to the girder. Diaphragm type D1 is provided with a simple bolted web shear connection to girders G2 and G3, while diaphragm type D2 is provided with a full moment connection using structural T’s to connect the diaphragm flange to the girders.

One type D1 diaphragm was instrumented. Strain gages (CH_1 and CH_2) were placed on the top and bottom flanges, two feet from the face of the web of girder G3. The total span of the diaphragm is 8 feet 6 inches. Figure 4.18 contains time-history plots of strain gages CH_1 and CH_2, with the test truck traveling in three different lanes: (a) inside northbound lane, (b) inside southbound, and (c) outside southbound. Also shown on the plots are the stress time-histories of the main girder gages, which can be used to identify the truck position in relation to the girder spans.

The behavior of the bridge is complicated by the fact that it consists of two separate north and south bound superstructures which are intermittently connected via the D1 and D2 diaphragms. Shown in Figure 4.18 are stress time-history plots for tests with the truck in different lanes. As can be seen, the behavior is complicated, and varies significantly with transverse truck position. However, it can be seen that the stresses are low.

It is important to note that although the diaphragm was detailed with “simple” connections (bolted web), there is moment transfer through the diaphragm between the north and south bound portions of the bridge.
(a) Truck in inside northbound lane

(b) Truck in inside southbound lane

(c) Truck in outside southbound lane

Figure 4.18 – Stress time histories at type D1 (pinned-pinned) diaphragm
Shown in Figure 4.19 are three similar stress time-histories for strain gages CH_5, CH_6, CH_7, and CH_8, with the test truck in the same three lanes: (a) inside northbound, (b) inside southbound, and (c) outside southbound. This diaphragm is a type D2 diaphragm, and is provided with full moment connections. As shown in the figure, CH_5 and CH_7 are on the top flange and CH_6 and CH_8 are on the bottom flange.

The behavior of the type D2 diaphragm is similar to that of the D1 diaphragm. The measured stresses are significantly lower, however, it should be noted that it is difficult to compare the stresses due to the differences in the connection details and placement of the strain gages.
Figure 4.19 – Stress time histories at type D2 (fixed-fixed) diaphragm

(a) Truck in inside northbound lane
*Pure global response (CH_5 and 6)*

(b) Truck in inside southbound lane
*Combination of global and local response*

(c) Truck in outside southbound lane
*Pure local response*
A summary of the maximum, minimum and stress range for each channel on the D1 diaphragm for each test is shown in Table 4.3. It can be seen that the highest stress ranges are produced when the test truck is in either the middle northbound or middle southbound lane.

<table>
<thead>
<tr>
<th>Truck in Lane</th>
<th>CH_1</th>
<th>CH_2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range</td>
<td>Max</td>
</tr>
<tr>
<td>3N</td>
<td>0.6</td>
<td>0.5</td>
</tr>
<tr>
<td>2N</td>
<td>0.8</td>
<td>0.6</td>
</tr>
<tr>
<td>1N</td>
<td>0.3</td>
<td>0.1</td>
</tr>
<tr>
<td>1S</td>
<td>0.4</td>
<td>0.3</td>
</tr>
<tr>
<td>2S</td>
<td>0.9</td>
<td>0.8</td>
</tr>
<tr>
<td>3S</td>
<td>0.7</td>
<td>0.6</td>
</tr>
</tbody>
</table>

\[ S_{R_{\text{max}}} = 0.9 \]

Table 4.3 – Summary of maximum, minimum, and stress ranges (in ksi) for strain gages on the top (CH_1) and bottom (CH_2) flange diaphragm type D1 for various test truck lane positions

*Note: Lane 1 is the inside lane, lane 3 is the outside lane*

A similar summary table for diaphragm type D2 is presented in Table 4.4. There is not a clear trend in the data, however, generally for southbound traffic, peak stresses are caused by load in the outside lane. For northbound traffic, the highest stresses were also caused by loads in the outside lane, however, comparable stresses were caused by loads in all lanes.

<table>
<thead>
<tr>
<th>Truck in Lane</th>
<th>CH_5</th>
<th>CH_6</th>
<th>CH_7</th>
<th>CH_8</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range</td>
<td>Max</td>
<td>Min</td>
<td>Range</td>
</tr>
<tr>
<td>3N</td>
<td>2.0</td>
<td>1.9</td>
<td>-0.1</td>
<td>1.3</td>
</tr>
<tr>
<td>2N</td>
<td>1.9</td>
<td>1.6</td>
<td>-0.3</td>
<td>1.3</td>
</tr>
<tr>
<td>1N</td>
<td>1.1</td>
<td>0.3</td>
<td>-0.8</td>
<td>1.8</td>
</tr>
<tr>
<td>1S</td>
<td>0.5</td>
<td>0.2</td>
<td>-0.3</td>
<td>0.7</td>
</tr>
<tr>
<td>2S</td>
<td>1.2</td>
<td>1.0</td>
<td>-0.1</td>
<td>1.4</td>
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<tr>
<td>3S</td>
<td>1.3</td>
<td>1.2</td>
<td>-0.1</td>
<td>1.6</td>
</tr>
</tbody>
</table>

\[ S_{R_{\text{max}}} = 2.0 \  1.8 \  2.1 \  1.6 \]

Table 4.4 – Summary of maximum, minimum, and stress ranges (in ksi) for strain gages on the top (CH_5, 7) and bottom (CH_6, 8) flange diaphragm type D2 for various test truck lane positions

*Note: Lane 1 is the inside lane, lane 3 is the outside lane*
4.3.5 Web Gap at Transverse Stiffener

At two locations, strain gages were placed on the girder web adjacent to the top and bottom web gaps at the transverse stiffener. The purpose of these gages was to quantify the web gap stresses at these locations. There is a diaphragm at the instrumented location.

Figure 4.20 contains a stress time-history of the four strain gages at the web gap for a crawl test with the truck in the outside southbound lane. It can be seen that when the truck is directly over the instrumented location, the stresses are maximum. At FB.40 (CH_11 and 12), the stresses are nearly equal and opposite at top and bottom. However, at FB.38 (CH_23 and 24), the stresses are very different. Since FB.38 is located in the negative moment region of girder G3, the transverse stiffener was not welded to the top flange, since it is in tension. On the other hand, FB.40 is located in the positive moment region and therefore the transverse stiffener was welded to the top flange. Due to the lack of a connection between the transverse stiffener and the top flange, it can be seen in Figure 4.20 that the stresses are higher at CH_23 than at CH_11 where there is a weld.

The stress fields at these locations have high gradients and are highly variable. However, it can be seen that the stresses result from global deformations of the bridge, since a reversal in the stress is evident when the test truck passes into the next span.

Figure 4.20 – Stress time-history for strain gages located within the web gap
CH_11, 12 – top & bottom G3 at FB.40
CH_23, 24 – top & bottom G3 at FB.38
A summary of the maximum, minimum and stress ranges for each channel, for each test is shown in Table 4.5. It can be seen that in general, the highest stress ranges are produced when the test truck is in the inside or middle northbound lane. Also, the highest stresses were observed at CH_23 where the transverse connection plate was not welded to the top flange.

<table>
<thead>
<tr>
<th>Truck in Lane</th>
<th>CH_11</th>
<th>CH_12</th>
<th>CH_23</th>
<th>CH_24</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range</td>
<td>Max</td>
<td>Min</td>
<td>Range</td>
</tr>
<tr>
<td>3N</td>
<td>0.4</td>
<td>0.1</td>
<td>-0.4</td>
<td>0.2</td>
</tr>
<tr>
<td>2N</td>
<td>0.4</td>
<td>0.0</td>
<td>-0.3</td>
<td>0.4</td>
</tr>
<tr>
<td>1N</td>
<td>1.0</td>
<td>0.0</td>
<td>-1.0</td>
<td>0.4</td>
</tr>
<tr>
<td>1S</td>
<td>0.3</td>
<td>0.2</td>
<td>0.0</td>
<td>0.4</td>
</tr>
<tr>
<td>2S</td>
<td>0.3</td>
<td>0.2</td>
<td>-0.1</td>
<td>0.3</td>
</tr>
<tr>
<td>3S</td>
<td>0.1</td>
<td>0.1</td>
<td>0.0</td>
<td>0.3</td>
</tr>
</tbody>
</table>

\[ S_{R_{\text{max}}} = 1.0 \quad 0.4 \quad 4.3 \quad 0.4 \]

Table 4.5 – Summary of maximum, minimum, and stress ranges (in ksi) for strain gages on the bottom flange of the four main girders for various test truck lane positions

Note: Lane 1 is the inside lane, lane 3 is the outside lane
4.3.6 Floorbeam Web and Top Flange Cope

Strain gages (CH_17 and 18) were placed horizontally, back-to-back on each side of the floorbeam web at the top flange cope where the floorbeam frames into the interior main girder. These gages were installed at floorbeam FB.39. At this location, a simple connection was provided between the floorbeam and girder G3. There is no diaphragm at this location. A very small fatigue crack was observed at this location.

Shown in Figure 4.21 is a stress time-history plot of the two strain gages. It can be seen that the response of at these locations is driven by local loading of the floorbeam. Highest stresses in the connection are generated when the test truck is directly over the floorbeam. As seen, the stresses are tension, as the top flange is pulling away from the main girder when the floorbeam is loaded. It can also be seen that the gages show an average stress of approximately 4.5 ksi, which represents the axial stresses component. There is also a bending component of approximately 1.5 ksi.
A summary of the maximum, minimum and stress ranges for each channel, for each test is shown in Table 4.6. It can be seen that the highest stress ranges are produced when the test truck is in the outside southbound lane. It is interesting to note that significant stresses were also observed when the test truck was in the middle southbound lane despite the fact that there is no diaphragm present at this floorbeam (FB39).

<table>
<thead>
<tr>
<th>Truck in Lane</th>
<th>CH_17</th>
<th></th>
<th></th>
<th>CH_18</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range</td>
<td>Max</td>
<td>Min</td>
<td>Range</td>
<td>Max</td>
<td>Min</td>
</tr>
<tr>
<td>3N</td>
<td>3.7</td>
<td>3.4</td>
<td>-0.3</td>
<td>7.4</td>
<td>6.4</td>
<td>-1.0</td>
</tr>
<tr>
<td>2N</td>
<td>4.0</td>
<td>3.7</td>
<td>-0.3</td>
<td>7.3</td>
<td>6.2</td>
<td>-1.1</td>
</tr>
<tr>
<td>1N</td>
<td>2.2</td>
<td>1.4</td>
<td>-0.7</td>
<td>4.0</td>
<td>3.2</td>
<td>-0.8</td>
</tr>
<tr>
<td>1S</td>
<td>0.4</td>
<td>0.2</td>
<td>-0.2</td>
<td>0.9</td>
<td>0.3</td>
<td>-0.6</td>
</tr>
<tr>
<td>2S</td>
<td>1.9</td>
<td>1.4</td>
<td>-0.5</td>
<td>4.5</td>
<td>3.6</td>
<td>-1.0</td>
</tr>
<tr>
<td>3S</td>
<td>0.3</td>
<td>0.1</td>
<td>-0.1</td>
<td>0.6</td>
<td>0.2</td>
<td>-0.3</td>
</tr>
</tbody>
</table>

\[ S_{R_{\text{max}}} = 4.0 \quad 7.4 \]

Table 4.6 – Summary of maximum, minimum, and stress ranges (in ksi) for strain gages on floorbeam web at top flange cope (back-to-back) for various test truck lane positions

*Note: Lane 1 is the inside lane, lane 3 is the outside lane*
4.3.7 Longitudinal Stiffener Terminations – Span 26

Figure 4.22 shows a stress time-history plot for the four strain gages installed at the termination of longitudinal stiffeners on girders G1, G2, and G3 in the tension/reversal region within span 26. Strain gages CH_25 and CH_28 are located on girders G1 and G3, respectively. Strain gages CH_26 and CH_27 are located back-to-back on girder G2.

The stress at the termination of the longitudinal stiffeners responds to global stress. Span 26 is part of a two-span unit consisting of spans 26 and 27. It can be seen that as the truck crosses span 27 first from the north, the measured stress is negative, but becomes positive when the truck crosses into span 26. Furthermore, with the truck in the outside lane, the stress in the girders decreases with increasing distance from the edge of the bridge, i.e., highest stress is observed at CH_25, the exterior girder, and lowest at CH_28, the most interior girder.

Figure 4.22 – Stress time-history for strain gages at termination of longitudinal stiffeners for crawl test in outside southbound lane
A summary of the maximum, minimum and stress ranges for each channel, for each test is shown in Table 4.7. It can be seen that as expected, the highest stress range in the exterior girder (CH_25) are produced when the test truck is in the outside southbound lane (3.1 ksi). Similarly, the largest stresses in the most interior girder, G3, were recorded with the truck in the inside northbound lane (1.2 ksi).

<table>
<thead>
<tr>
<th>Truck in Lane</th>
<th>CH_25</th>
<th>CH_26</th>
<th>CH_27</th>
<th>CH_28</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range</td>
<td>Max</td>
<td>Min</td>
<td>Range</td>
</tr>
<tr>
<td>3N</td>
<td>0.5</td>
<td>0.2</td>
<td>-0.2</td>
<td>0.3</td>
</tr>
<tr>
<td>2N</td>
<td>0.5</td>
<td>0.2</td>
<td>-0.3</td>
<td>0.8</td>
</tr>
<tr>
<td>1N</td>
<td>0.8</td>
<td>0.3</td>
<td>-0.4</td>
<td>1.1</td>
</tr>
<tr>
<td>1S</td>
<td>1.4</td>
<td>0.8</td>
<td>-0.6</td>
<td>1.4</td>
</tr>
<tr>
<td>2S</td>
<td>2.3</td>
<td>1.5</td>
<td>-0.8</td>
<td>1.2</td>
</tr>
<tr>
<td>3S</td>
<td>3.1</td>
<td>2.1</td>
<td>-1.0</td>
<td>0.9</td>
</tr>
</tbody>
</table>

$S_{R\text{max}} = 3.1 \quad 1.4 \quad 2.2 \quad 1.2$

Table 4.7 – Summary of maximum, minimum, and stress ranges (in ksi) for strain gages at the termination of the lower longitudinal stiffener of girders G1 (CH_25), G2 (CH_26, 27), and G3 (CH_28) for various test truck lane positions.

Note: Lane 1 is the inside lane, lane 3 is the outside lane.
4.4 Long-term Monitoring

The long-term monitoring of spans 25 and 26 was conducted from March 24 to April 22, 2004, for a total of 29.1 days of long-term data. Based on a review of the controlled load test data, 24 of the 28 channels used during the controlled load tests were selected for long-term monitoring. Data in all monitored channels were recorded based on three predefined trigger events: (1) a stress of 1.75 ksi was exceeded in CH_3; (2) a stress of 1.5 ksi was exceeded in CH_14; and (3) a stress of 1.9 ksi was exceeded in CH_4. Trigger event (1) represents southbound traffic in the outside lane. Trigger event (2) represents north or southbound traffic in the inside lanes. Finally, trigger event (3) represents northbound traffic in the outside lane. Eight seconds of data were taken for each trigger event. The magnitudes of the trigger thresholds were based on a review of the data collected during the controlled load tests and on data collected as random traffic crossed the span while on site.

4.4.1 Results of Long-term Monitoring

In this section, results of the fatigue evaluation of spans 25 and 26 are presented. The rainflow cycle counting method was used to determine the stress-range histograms. Each detail type where monitoring was performed is analyzed and presented separately below. An in-depth discussion of the methodology used for the fatigue evaluation can be found in Appendix B.

4.4.2 Main Girders

Figure 4.23 contains stress-range histograms for the four strain gages located on the bottom flange of the four main girders of span 25. Gages CH_3, 13, 14, and 4 are located on girders G1, G2, G3, and G4, respectively. The histograms for the four gages appear to be consistent.

These locations are conservatively assumed to be classified as category C, because of the transverse fillet welds connecting the transverse stiffeners to the flanges. All cycles less than 2.5 ksi were discarded, as indicated in Figure 4.23. Though these details are located inside the flange and the measured stress is on the outside of the flange, the difference between the two should be minimal due to the large depth of the girders with respect to the flange thickness. Table 4.8 contains the estimates of the remaining life at these details. As shown the effective stress ranges and the number of cycles accumulated per day are low. Therefore, these details are assumed to have infinite life.
Figure 4.23 – Stress-range histogram for strain gages located on the bottom flanges of the four main girders

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>$S_{\text{Rmax}}$ (ksi)</th>
<th>Cycles &gt; CAFL</th>
<th>$S_{\text{Ref}}$ (ksi)</th>
<th>Cycles/day</th>
<th>Remaining Life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_3</td>
<td>3.5</td>
<td>0</td>
<td>0.00%</td>
<td>2.8</td>
<td>1</td>
</tr>
<tr>
<td>CH_4</td>
<td>3.5</td>
<td>0</td>
<td>0.00%</td>
<td>2.8</td>
<td>3</td>
</tr>
<tr>
<td>CH_13</td>
<td>3.0</td>
<td>0</td>
<td>0.00%</td>
<td>2.8</td>
<td>0.1</td>
</tr>
<tr>
<td>CH_14</td>
<td>3.5</td>
<td>0</td>
<td>0.00%</td>
<td>2.8</td>
<td>0.4</td>
</tr>
</tbody>
</table>

Table 4.8 – Summary of life calculations for main girders
4.4.3 Transverse Stiffener - Flange Connection

Figure 4.24 contains the stress-range histograms for the strain gages mounted to the transverse stiffeners at the toe of the stiffener-to-flange weld toe. Strain gages CH_9 and CH_10 are back-to-back at a location with a diaphragm, whereas CH_15 and CH_16 are located back-to-back where there is no diaphragm.

Table 4.9 presents a summary of the life calculations based on the data from these strain gages. These details were assumed to be classified as category C. Stress cycles less than 2.5 ksi were discarded, as indicated in Figure 4.24. It can be seen that a large percentage (over 3%) of stress cycles exceed the CAFL of the detail, equal to 10.0 ksi. This is larger than a frequency of 1/10,000, accepted as the limit for infinite life. Therefore, the detail is estimated to have finite life. Based on the rate and range of stress cycles observed during the monitoring, and assuming this is representative of the entire life of the structure, the data indicate that these details have a negative life, i.e. crack should have already occurred. This is the case as this cracking has been identified on the bridge as type “K” cracking.

![Stress-range histogram for strain gages mounted to the transverse stiffener adjacent to the stiffener-to-flange weld toe](image)

**Figure 4.24** – Stress-range histogram for strain gages mounted to the transverse stiffener adjacent to the stiffener-to-flange weld toe
Table 4.9 – Summary of life calculations for transverse stiffener-to-girder flange welds

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>$S_{R_{\text{max}}}$ (ksi)</th>
<th>Cycles &gt; CAFL</th>
<th>$S_{\text{Ref}}$ (ksi)</th>
<th>Cycles/ Remaining Life (days)</th>
<th>Remaining Life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_9</td>
<td>16.0</td>
<td>469</td>
<td>0.59%</td>
<td>5.1</td>
<td>2,730</td>
</tr>
<tr>
<td>CH_10</td>
<td>11.5</td>
<td>3</td>
<td>0.01%</td>
<td>3.9</td>
<td>1,427</td>
</tr>
<tr>
<td>CH_15</td>
<td>16.0</td>
<td>3,604</td>
<td>2.69%</td>
<td>5.8</td>
<td>4,613</td>
</tr>
<tr>
<td>CH_16</td>
<td>16.0</td>
<td>1,616</td>
<td>1.68%</td>
<td>5.7</td>
<td>3,306</td>
</tr>
</tbody>
</table>

4.4.4 Diaphragms

Figure 4.25 contains the stress-range histograms for six gages mounted to both type D1 (CH_1 and 2) and D2 (CH_5, 7, 19, and 21) diaphragms. These details were assumed to be classified as category B, because of the bolted connections, and the fact that there are no transverse stiffeners. Since the peak stress at any of these strain gages was 4 ksi, which is equal to 1/4 the CAFL for this detail, it can be assumed that infinite life can be expected.

Figure 4.25 – Stress-range histogram for strain gages mounted to type D1 (CH_1, 2) and D2 (CH_5, 7, 19, 21) diaphragms
4.4.5 Web Gap at Transverse Stiffener

Figure 4.26 contains the stress-range histograms for the strain gages mounted in the web gap at the transverse connection plate at the floorbeam. This detail was assumed to be classified as category C. A stress range cutoff of 2.5 ksi was used, as indicated in Figure 4.26.

As shown in Table 4.5, with the exception of strain gage CH_23, the stresses ranges and number of cycles are low. However, the CAFL of the detail was never exceeded and therefore, infinite life can be expected. Highest stress at this location can be expected at the top flange. Furthermore, since the transverse stiffener is not welded to the girder flange at CH_23, the highest stresses were recorded there. Table 4.10 presents a summary of the life calculations based on the data from strain gage CH_23.

![Stress-range histogram for strain gages mounted in the web gap at the transverse connection plate at the floorbeam-girder connection](image)

Figure 4.26 – Stress-range histogram for strain gages mounted in the web gap at the transverse connection plate at the floorbeam-girder connection

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>$S_{Rmax}$ (ksi)</th>
<th>Cycles &gt; CAFL</th>
<th>$S_{Reff}$ (ksi)</th>
<th>Cycles/ Remaining Life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_23</td>
<td>9.5</td>
<td>0</td>
<td>3.37</td>
<td>881</td>
</tr>
</tbody>
</table>

Table 4.10 – Summary of life calculations for web gap strain gage CH_23
(Note other web gap gage data also yield infinite life)
4.4.6 Floorbeam Web and Top Flange Cope

Presented in Figure 4.27 is the stress-range histograms for the two gages mounted horizontally back-to-back to the web of the floorbeam at the top flange cope where the floorbeam frames into an interior main girder.

These details are assumed to be classified as category E. A stress range cutoff of 1.0 ksi was used, as indicated in Figure 4.27. As can be seen, there is a significant difference between the histograms of the two gages, even though they were installed back-to-back.

Table 4.11 contains a summary of the life calculations for the two gages. At both strain gages, the peak stresses exceeded the CAFL with a frequency of more than 1/10,000. Therefore, the details are determined to have finite life. However, there is a large discrepancy in the life calculations. Strain gage CH_18 indicates that the detail would have cracked, whereas CH_17 indicates that there is roughly 30 years of life remaining. In actuality, this detail has cracked. The high stress gradient in this area is likely the cause of the scatter in the results.

![Stress-range histogram for strain gages mounted to the floorbeam web at the top flange cope](image)

Figure 4.27 - Stress-range histogram for strain gages mounted to the floorbeam web at the top flange cope
### Table 4.11 - Summary of life calculations for strain gages mounted to the floorbeam web at the top flange cope

<table>
<thead>
<tr>
<th>Gage</th>
<th>Strain (ksi)</th>
<th>$S_{R_{max}}$</th>
<th>Cycles &gt; CAFL</th>
<th>$S_{Reff}$ (ksi)</th>
<th>Cycles/day</th>
<th>Remaining Life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_17</td>
<td>7.5</td>
<td>811</td>
<td>0.56%</td>
<td>2.2</td>
<td>5,002</td>
<td>29</td>
</tr>
<tr>
<td>CH_18</td>
<td>12.0</td>
<td>39154</td>
<td>2.78%</td>
<td>2.2</td>
<td>48,462</td>
<td>-24</td>
</tr>
</tbody>
</table>

4.4.7 **Longitudinal Stiffener Terminations – Span 26**

Shown in Figure 4.28 is a stress-range histogram for the strain gages mounted at the termination of the lower longitudinal stiffeners of the main girders in span 26. Strain gages CH_25 and 28 were mounted to girders G1 and G3, respectively. Strain gages CH_26 and 27 were mounted back-to-back on girder G2.

These details are assumed to be classified as category E. A stress range cutoff of 1.0 ksi was used, as indicated in Figure 4.28. As can be seen, there is a significant difference between the histograms of the four gages. Table 4.12 presents the life calculations for these four gages. With the exception of strain gage CH_25, the data from all the gages predict infinite life at these details. Strain gage CH_25 predicts finite life of greater than 150 years, which is essentially infinite.

![Stress-range histogram for strain gages mounted to the web of the main girders in span 26 at the termination of the longitudinal stiffeners](image-url)
<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>( S_{R\text{max}} ) (ksi)</th>
<th>Cycles &gt; CAFL</th>
<th>( S_{\text{Ref}} ) (ksi)</th>
<th>Cycles/Day</th>
<th>Remaining Life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_25</td>
<td>6.5</td>
<td>22</td>
<td>0.03%</td>
<td>1.8</td>
<td>2,457</td>
</tr>
<tr>
<td>CH_26</td>
<td>3.5</td>
<td>0</td>
<td>0.00%</td>
<td>1.3</td>
<td>633</td>
</tr>
<tr>
<td>CH_27</td>
<td>6.0</td>
<td>2</td>
<td>0.00%</td>
<td>1.6</td>
<td>2,243</td>
</tr>
<tr>
<td>CH_28</td>
<td>4.5</td>
<td>0</td>
<td>0.00%</td>
<td>1.3</td>
<td>205</td>
</tr>
</tbody>
</table>

Table 4.12 - Summary of life calculations for strain gages mounted to the web of the main girders in span 26 at the termination of the longitudinal stiffeners.
5.0 Tied Arch

5.1 Bridge Description – Tied Arch
The tied arch span of the Neville Island bridge has a main span of 750 feet. The ties are box girders approximately 12 feet 6 inches in depth, and 3 feet 6 inches in width. The span between the steel box arches is 110 feet. The ties are suspended from the arch at 50 foot intervals. The welded plate girder floorbeams are 10 feet in depth and are located at every suspender.

5.2 Summary of Instrumentation Layout – Tied Arch
The following section summarizes the instrumentation plan. A detailed instrumentation plan is included in Appendix A.

5.2.1 Tie Girders
Both tie girders of the bridge were instrumented. Gages were installed to measure both global response of the girders and the local response of key welded details within the tie girders. These gages will be described in the following sections.

5.2.1.1 Global Tie Stresses
On the east tie, four strain gages were installed at a section 5 feet 10 1/2 inches north of panel point 12. Two gages were mounted to the top flange, 1 inch from the edge of the flange, while the other two gages were mounted directly below on the bottom flange. Figure 5.1 shows the top side of the east girder (CH_14 and CH_15) and Figure 5.2 shows the underside of the east girder (CH_16 and CH_17).

Figure 5.1 – Strain gages mounted to the top flange of the east tie girder
5’-10 1/2” north of PP12
view looking south
Figure 5.2 – Strain gages mounted to the bottom flange of the east tie girder  
5’-10 1/2” north of PP12  
(view looking north)

Only two gages were installed on the west tie girder. Figure 5.3 shows CH_11 on 
the top flange of the tie girder. CH_12 was located on the bottom flange (directly below 
CH_11).
Figure 5.3 – Strain gage mounted to the top flange of the west tie girder
5’-10 1/2” north of PP12
5.2.1.2 Diaphragm Stiffeners

At two locations, strain gages were placed on the tie girder web at the end of diaphragm stiffeners. Matching gages were also placed on the diaphragm stiffener itself. An example is shown in Figure 5.4 (CH_24 and CH_25). Installation of CH_20 and CH_21 installed at PP 13 was similar. In most cases, gages were placed as close as possible to the toe of the weld. Note however that CH_24 was placed against the stiffener/web weld, but equidistant from the diaphragm and the free edge of the stiffener (see Figure 5.4).

Figure 5.4 – Strain gages at diaphragm at PP 11
5.2.1.3 Backing Bar

At three locations, strain gages were installed on the backing bar used at the tie girder web-to-flange weld where the bar passes through the sealing diaphragm. The gages were placed as close as possible to the toe of the bar/diaphragm fillet weld. Figure 5.5 contains a photograph of a typical installation. Note that one gage is placed on the backing bar (CH_26), while one is placed on the bottom flange of the girder (CH_27).

Figure 5.5 – Strain gages on backing bar and bottom flange of the tie girder at PP 11
5.2.1.4 Longitudinal Stiffeners

At one location a strain gage was installed on the longitudinal web stiffener at the toe of the weld to the sealing diaphragm, as shown in Figure 5.6. Additionally, a single strain gage was mounted to the tie girder web at the termination of the stiffener. This is shown in Figure 5.7.

![Figure 5.6 – Strain gage mounted to longitudinal web stiffener at the stiffener/sealing diaphragm weld toe](image1)

![Figure 5.7 – Strain gage mounted to interior face of the east web of the upstream tie girder at the termination of the longitudinal stiffener](image2)
5.2.1.5 Web Gap at Bulkhead
A strain gage was installed in the tie girder web gap at the primary diaphragm, at the centerline of PP14. The strain gage was mounted in a vertical orientation to measure the out-of-plane bending stresses induced into the web by flexure in the floorbeam. The location of this strain gage can be seen in Figure 5.7 above.

5.2.1.6 Sealing Diaphragm Access Hole
A horizontally oriented strain gage was installed above the access hole, adjacent to the edge of the hole. This can be seen in Figure 5.8. This strain gage was installed to investigate the feasibility of making the access holes bigger. In their current configuration, the access holes hinder inspection because they are very small and difficult to pass through. It was found that the live load stress ranges were small. Therefore, the hole could be enlarged without any adverse effects.

![Figure 5.8 – Strain gage mounted to sealing diaphragm above access hole](image)

5.2.2 Floorbeam
Two strain gages were installed on the floorbeam at panel point 14, the first interior floorbeam on the north side of the bridge. Strain gage CH_1 was mounted to the web of the floorbeam at the termination of the gusset plate which receives the lateral bracing members. Strain gage CH_3 was mounted to the floorbeam web in a vertical orientation against the web/flange weld toe and adjacent to the web gap formed at the transverse stiffener cope.
5.2.3 Hangers
Three hangers were instrumented with strain gages. The gages were mounted to the exterior hanger plate which connects (along with an inner plate) the suspenders to the tie girders. At panel point 14, the hangers at the east and west tie girders were instrumented. At the east hanger, both the inside and outside of the plate were gaged to determine if there is any out-of-plane bending of the plate, and to resolve the axial stress component in the plate. At the west girder, only the outer surface was gaged. An additional hanger plate was instrumented at the west tie girder at panel point 14. A photograph of a typical hanger gage is shown in Figure 5.9.

Figure 5.9 – Strain gage mounted to outer surface of hanger plate at PP14
5.2.4 Accelerometers

Three accelerometers were mounted to the bridge to investigate its dynamic properties and to assess the effect these properties might have on the fatigue performance of the bridge. Two vertically oriented accelerometers were installed at each tie girder at panel point 11. At the east tie girder, a horizontally oriented accelerometer was installed (also at panel point 11). This point was chosen because it will allow the maximum number of vibration modes to be measured. Figure 5.10 shows the vertical accelerometer installed at the west side of the bridge.

Figure 5.10 – Vertically oriented accelerometer mounted to the west suspender end block at PP 14
5.2.5 Anemometer

An anemometer was mounted to the east side of the floorbeam at panel point 12. Wind speed and direction were recorded and used to trigger the data logger during periods of high wind speed. This triggered data is used to assess the behavior of the bridge during high wind events. It was noted during on-site monitoring at the Birmingham Bridge, a similar tied-arch bridge, that strong winds excited low modes of vibration. Because of low damping, a large number of stress cycles were accumulated due to high winds. These stress cycles were in addition to those caused by normal traffic. Therefore, the main reason for installing this anemometer was to see if this phenomena occurs on the Neville Island Bridge when high winds are observed. A photograph of the anemometer is shown in Figure 5.11.

Figure 5.11 – Anemometer mounted to the east side of the floorbeam on PP 12
5.3 Results of Controlled Load Tests - Tied Arch

The results from the field work on the tied arch span of the Neville Island Bridge are presented in this section. Specifically, the global and local behavior of the bridge is examined, utilizing data collected during the controlled load tests. The instrumentation plans for the tied arch are provided in Appendix A.

5.3.1 General Response

As previously discussed, the portion of the bridge discussed in this section is a tied arch with a main span of 750 feet. The primary structure consists of two 12 foot 6 inch steel box tie girders which are suspended from a pair of steel arches via wire rope suspenders arranged on 16 panel points at 50 feet on center. The arches are spaced at 110 feet from center to center.

The floor system consists of floorbeams, stringers which rest on the top flange of the floorbeams, and an 8 inch concrete deck. The plate girder floorbeams span between the tie girders at each panel point. The stringers are 33 inch deep rolled steel stringers at 7 foot 1 inch spacing. The end floorbeams are box girders rather than “I” shapes.

In general, the observed response of the bridge was driven by live loading. Wind speed and direction was recorded throughout the monitoring period. However, during events of high wind speed, the stresses in the bridge members were not noticeably different than during periods of calm winds.

As will be discussed in further detail in the sections below, the tie girders respond primarily in bending. Observed axial response was significantly less. Therefore, rather than acting primarily as a tension tie for the arch under live load, the girder carries a significant percentage of the load to the supports as a flexural element. The behavior can be compared to a beam on an elastic foundation since there are elastic supports at each of the suspender cables.

5.3.2 Tie Girders

5.3.2.1 Global Tie Behavior

As discussed previously, longitudinally oriented strain gages were installed on both the east and west tie girders 5 feet 10 ½ inches to the north of panel point 12. On the east tie girder, four gages were installed (CH_14 and CH_15 on the top flange, CH_16 and CH_17 on the bottom flange). On the west tie, only two gages were installed on the centerline of the tie, (CH_11 on the top flange and CH_12 on the bottom flange.)

Figure 5.12 contains a typical plot of the response of the east tie girder during a crawl test with the test truck in the center north-bound lane. The west tie response is very similar. As can be seen in the plot, the truck took 80 seconds to cross the tied arch span. Therefore, the average speed of the truck was 750 ft/80 sec = 9.4 ft/sec, or 6.4 mph. If it is assumed that the truck moved at a near-constant velocity, the plot shown in the figure represents an influence line for the tie girder at PP 12. It can be seen that when the truck is positioned on the south end of the bridge, it causes negative bending in the tie at PP 12 (i.e. tension in the top flange, CH_14, CH_15). After it crosses a certain point, the truck causes positive bending in the tie at PP 12 (i.e. tension in the bottom flange, CH_16, CH_17). When the truck is positioned at the gage location, near PP12, the stress is maximum.
Figure 5.12 also demonstrates that the tie is dominated by bending, despite the fact that a tie girder is normally thought of as a tension member. Also, since the tie is “hung” from the steel arch by suspender cables, the tie behaves as a beam on an elastic foundation. It is noted that the tie girder is in tension under dead load to resist the outward thrust of the arch.

Figure 5.12 – Stress time-history for gages on the east tie girder 5’-10 1/2” north of PP 12 for crawl test with north bound truck in middle lane

The bending component of the measured stress in the tie can be determined by taking half the difference between the top and bottom flange stresses. The axial component is calculated as the average of the top and bottom flange stress. The axial and bending components are plotted in Figure 5.13. Note that bending stress is shown for the top flange (i.e. positive stress means tension in the top flange or negative bending.) Again it can be seen that the stress in the tie girder is dominated by bending. The peak bending stress is -0.8 ksi while the peak axial stress is 0.1 ksi tension. Furthermore, it is evident in the figure that the axial stress in the tie reaches a peak value when the truck is closer to the center of the arch. Table 5.1 contains a summary of the peak measured axial and bending stresses in the tie girders.
Figure 5.13 – Stress time-history for calculated bending and axial stress on east tie girder 5’-10 1/2” north of PP 12 for crawl test with north bound truck in middle lane.
Taking the average of the downstream gages and subtracting it from the average of the upstream gages yields the out-of-plane bending stress in the tie. This is plotted versus time in Figure 5.14. It can be seen that the magnitude of the out-of-plane stresses are very small compared to the axial and in-plane bending stresses. In this plot, the stress represents the weak axis bending stress in the outside web plate. Note that when the load is positioned at PP 12, there is a slight amount of weak axis bending on the tie girder which places the outer web into compression.

![Stress time-history for calculated out-of-plane bending stress on east tie girder 5'-10 1/2" north of PP 12 for crawl test with north bound truck in middle lane](image)

Figure 5.14 – Stress time-history for calculated out-of-plane bending stress on east tie girder 5'-10 1/2” north of PP 12 for crawl test with north bound truck in middle lane

It was noted that the stresses in the tie girders varied with lane position of the test truck. When the test truck was closest to the girder under consideration, the stresses (axial and bending) were maximum. The stresses reduced with increasing distance between the test truck lane position and the tie girder. Table 5.1 presents the peak stresses in the tie girder for all the lane positions. It can be seen that there is fairly good symmetry in the results for the east and west tie girders. For example, the maximum positive bending stresses in the east tie vary from 0.47 ksi to 0.36 ksi from 3S to 3N, and for the west tie, these values range from 0.42 to 0.29 ksi, from 3N to 3S.
Table 5.1 – Summary of peak measured axial and bending stresses (in ksi) in the west and east tie girders for the various test truck lane positions

Note: Lane 1 is the inside lane, lane 3 is the outside lane

5.3.2.2 Diaphragm Stiffeners

Figure 5.15 contains a stress time-history plot for strain gages CH_24 and CH_25 located on the diaphragm stiffener and on the girder web adjacent to the end of the stiffener weld. It can be seen that the stress history tracks the trend of the tie girder stress. Note that the tie stress is given at panel point 12 while CH_24 and CH_25 are at panel point 11. As expected, the stress in the web at the termination is much higher than the stress in the stiffener since the stiffener is a relatively short attachment to the web.

Figure 5.15 – Stress time-history for stress in (CH_24) and at the termination of (CH_25) diaphragm stiffeners for crawl test with north bound truck in outside lane

Note: Stress in the tie girder at PP 12 is shown for reference
Table 5.2 contains a summary of the maximum, minimum, and stress ranges for strain gages CH_21, CH_22, CH_24, and CH_25 mounted at the diaphragm stiffener. Note that the peak stresses were recorded when the test truck was in the outside northbound lane.

<table>
<thead>
<tr>
<th>Truck in Lane</th>
<th>CH_20</th>
<th></th>
<th></th>
<th>CH_21</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range</td>
<td>Max</td>
<td>Min</td>
<td>Range</td>
<td>Max</td>
<td>Min</td>
</tr>
<tr>
<td>3N</td>
<td>0.4</td>
<td>0.3</td>
<td>-0.1</td>
<td>1.6</td>
<td>1.3</td>
<td>-0.3</td>
</tr>
<tr>
<td>2N</td>
<td>0.4</td>
<td>0.3</td>
<td>-0.1</td>
<td>1.5</td>
<td>1.2</td>
<td>-0.4</td>
</tr>
<tr>
<td>1N</td>
<td>0.4</td>
<td>0.3</td>
<td>-0.1</td>
<td>1.4</td>
<td>1.0</td>
<td>-0.4</td>
</tr>
<tr>
<td>1S</td>
<td>0.4</td>
<td>0.2</td>
<td>-0.2</td>
<td>1.5</td>
<td>0.8</td>
<td>-0.7</td>
</tr>
<tr>
<td>2S</td>
<td>0.3</td>
<td>0.2</td>
<td>-0.1</td>
<td>1.0</td>
<td>0.8</td>
<td>-0.3</td>
</tr>
<tr>
<td>3S</td>
<td>0.2</td>
<td>0.1</td>
<td>-0.1</td>
<td>0.9</td>
<td>0.6</td>
<td>-0.3</td>
</tr>
</tbody>
</table>

\[ S_{R_{\text{max}}} = 0.4 \quad 1.6 \]

<table>
<thead>
<tr>
<th>Truck in Lane</th>
<th>CH_24</th>
<th></th>
<th></th>
<th>CH_25</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range</td>
<td>Max</td>
<td>Min</td>
<td>Range</td>
<td>Max</td>
<td>Min</td>
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<td>0.6</td>
<td>0.4</td>
<td>-0.2</td>
<td>2.1</td>
<td>1.6</td>
<td>-0.5</td>
</tr>
<tr>
<td>2N</td>
<td>0.5</td>
<td>0.4</td>
<td>-0.1</td>
<td>1.9</td>
<td>1.4</td>
<td>-0.5</td>
</tr>
<tr>
<td>1N</td>
<td>0.5</td>
<td>0.3</td>
<td>-0.1</td>
<td>1.8</td>
<td>1.2</td>
<td>-0.5</td>
</tr>
<tr>
<td>1S</td>
<td>0.5</td>
<td>0.2</td>
<td>-0.3</td>
<td>2.0</td>
<td>1.0</td>
<td>-1.0</td>
</tr>
<tr>
<td>2S</td>
<td>0.3</td>
<td>0.2</td>
<td>-0.1</td>
<td>1.3</td>
<td>0.9</td>
<td>-0.4</td>
</tr>
<tr>
<td>3S</td>
<td>0.3</td>
<td>0.2</td>
<td>-0.2</td>
<td>1.2</td>
<td>0.7</td>
<td>-0.5</td>
</tr>
</tbody>
</table>

\[ S_{R_{\text{max}}} = 0.6 \quad 2.1 \]

Table 5.2 – Summary of maximum, minimum, and stress ranges (in ksi) for strain gages mounted at the diaphragm stiffener for the various test truck lane positions

CH_20 & CH_21 are at PP 13
CH_24 & CH_25 are at PP 11

Note: Lane 1 is the inside lane, lane 3 is the outside lane
5.3.2.3 Backing Bars

Figure 5.16 contains the stress-time history for two gages at the backing bar detail used at the tie girder web-to-flange full penetration weld. Shown in the figure are the response of three strain gages at panel point 12: CH_18 located on the backing bar, CH_19 located on the bottom flange adjacent to the backing bar, and the stress measured by CH_16 on the bottom of the bottom flange of the tie girder, for reference. It can be seen that there is very little difference between the measured response of the three strain gages, indicating that there is low stress concentration at this detail. It also indicates that the backing bar acts compositely with the tie girder since compatibility is provided by the continuous welds along its length.

Figure 5.16 – Stress time-history for stress in backing bar (CH_18) and in the tie girder bottom flange adjacent to the backing bar (CH_19) at PP12 for crawl test with north bound truck in outside lane

Note: Stress in the tie girder at PP 12 is shown for reference
Shown in Table 5.3 are the minimum, maximum, and stress ranges for all strain gages mounted at the backing bar detail (CH_18, CH_19, CH_23, CH_26, & CH_27). Note that peak stresses occur when the test truck was in the outside northbound lane. Furthermore, the stresses in the flange and back up bar are very similar.

<table>
<thead>
<tr>
<th>Truck in Lane</th>
<th>CH_18 (on BU bar)</th>
<th>CH_19 (on flange)</th>
<th>CH_23 (on BU bar)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range</td>
<td>Max</td>
<td>Min</td>
</tr>
<tr>
<td>3N</td>
<td>1.4</td>
<td>1.0</td>
<td>-0.5</td>
</tr>
<tr>
<td>2N</td>
<td>1.4</td>
<td>1.0</td>
<td>-0.5</td>
</tr>
<tr>
<td>1N</td>
<td>1.3</td>
<td>0.8</td>
<td>-0.5</td>
</tr>
<tr>
<td>1S</td>
<td>1.2</td>
<td>0.7</td>
<td>-0.4</td>
</tr>
<tr>
<td>2S</td>
<td>1.1</td>
<td>0.7</td>
<td>-0.4</td>
</tr>
<tr>
<td>3S</td>
<td>0.9</td>
<td>0.5</td>
<td>-0.3</td>
</tr>
</tbody>
</table>

$S_{R_{max}} = 1.4 \quad 1.7 \quad 1.2$

<table>
<thead>
<tr>
<th>Truck in Lane</th>
<th>CH_26 (on BU bar)</th>
<th>CH_27 (on flange)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range</td>
<td>Max</td>
</tr>
<tr>
<td>NB</td>
<td>1.3</td>
<td>0.9</td>
</tr>
<tr>
<td>NB</td>
<td>1.1</td>
<td>0.8</td>
</tr>
<tr>
<td>NB</td>
<td>1.1</td>
<td>0.7</td>
</tr>
<tr>
<td>SB</td>
<td>1.2</td>
<td>0.6</td>
</tr>
<tr>
<td>SB</td>
<td>0.3</td>
<td>0.5</td>
</tr>
<tr>
<td>SB</td>
<td>0.7</td>
<td>0.4</td>
</tr>
</tbody>
</table>

$S_{R_{max}} = 1.3 \quad 1.3$

Table 5.3 – Summary of maximum, minimum, and stress ranges (in ksi) for strain gages mounted at the diaphragm stiffener for the various test truck lane positions

CH_18 & CH_19 are at PP 12

CH_23 is at PP 13

CH_26 & CH_27 are at PP 11

Note: Lane 1 is the inside lane, lane 3 is the outside lane
5.3.2.4 Longitudinal Stiffeners

Stress time-history plots for the longitudinal stiffener details are shown in Figure 5.17. Strain gage CH_7 is located on the outer web at the termination of the longitudinal stiffener at panel point 14. Strain gage CH_9 is located on the longitudinal stiffener (on the outer web) where the stiffener meets the sealing diaphragm. Both gages were placed against the weld toe.

It can be seen in the figure that the trends at these gages track the tie girder stress. Furthermore, it can be seen that the level of stress is low. That is, the magnitude of the stress concentration at these details is low, because the stress in at the detail is of comparable magnitude to that of the girder. Also, the stresses are lower since these details are located further up the web of the girder and are therefore closer to the neutral axis.

Figure 5.17 – Stress time-history for stress in the web at the termination of the longitudinal stiffener (CH_7) and in the longitudinal stiffener at the intersection with the sealing diaphragm(CH_9) at PP14 for crawl test with north bound truck in outside lane

Note: Stress in the tie girder at PP 12 is shown for reference
5.3.2.5 Web Gap at Bulkhead

Figure 5.18 contains a stress time-history for the vertically oriented strain gage located in the web gap between the bottom flange and the bulk head at panel point 14, on the outer web, CH_10. The stress in the web gap is low, and is not cause for concern as web gap cracking is not expected. The previously installed retrofits are performing as intended.

Figure 5.18 – Stress time-history for vertical stress in the web web gap between the bottom flange and bulkhead at PP14 for crawl test with north bound truck in outside lane

Note: Stress in the tie girder at PP 12 is shown for reference
5.3.2.6 Sealing Diaphragm Access Hole

A strain gage (CH_22) was installed at the sealing diaphragm access hole at panel point 13. The gage was oriented horizontally above the hole. Figure 5.19 contains a stress time-history for this gage. The stresses are extremely low around the opening for the access hole. The stress is maximum (in tension) when the truck is directly over panel point 13.

This strain gage was installed to obtain information on the magnitude of the live load stresses carried in the sealing diaphragm. The sealing diaphragms are very difficult to pass through and as a result, make inspection difficult. Since the magnitude of the live load stress is very low, the opening could be enlarged with no negative effects on the performance of the bridge.

Figure 5.19 – Stress time-history for horizontal stress in the sealing diaphragm above the access hole at PP13 for crawl test with north bound truck in outside lane

Note: Stress in the tie girder at PP 12 is shown for reference
5.3.3 Floorbeam

Two strain gages were installed on the floorbeam at panel point 14, the first interior floorbeam on the north side of the bridge. The first gage, CH_1, was installed at the end of a longitudinal gusset plate to which the lateral bracing is connected. The second gage, CH_3, was installed vertically on the web alongside the web gap at the transverse stiffener cope. The gap was not long enough for the gage to be installed within the gap. These gages are located close to the mid span of the floorbeam.

Strain gage CH_1 measures primary bending stress with additional stress due to the termination of the gusset plate. Therefore the stress measured by this gage is maximum when the test truck is over the mid span of the floorbeam, since it behaves as a simple span.

A stress time-history plot for strain gages CH_1 and CH_3, for a northbound test truck in lane 1 (inside lane), is presented in Figure 5.20. It can be seen from the plot that the influence of the truck moving onto the bridge at PP 0 can be seen at strain gage CH_1 as vibration. The frequency of this vibration is approximately 2.9 Hz. Prior to reaching PP 14, there is a slight reversal in stress, probably due to the 3-span continuous stringers. However, the maximum stress occurs when the test truck is between PP13 and PP15, which causes primary tension near the bottom flange, where CH_1 is located.

Strain gage CH_3 records any out-of-plane bending of the web adjacent to the web gap. It can be seen from Table 5.4 and Figure 5.20 that the stress measured at CH_3 was small.

![Stress Time History for Floorbeam Strain Gages](image_url)
A summary of the maximum, minimum and stress ranges for strain gages CH_1 and CH_3, for each of the test truck lane positions, is presented in Table 5.4. It can be seen that the maximum stress at CH_1 was 2.1 ksi (maximum range of 2.4 ksi), and that the stress drops as the test truck moved to the outer lanes.

<table>
<thead>
<tr>
<th>Truck in Lane</th>
<th>CH_1</th>
<th>CH_3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range</td>
<td>Max</td>
</tr>
<tr>
<td>1N</td>
<td>2.4</td>
<td>2.1</td>
</tr>
<tr>
<td>1S</td>
<td>2.2</td>
<td>2.0</td>
</tr>
<tr>
<td>2N</td>
<td>1.6</td>
<td>1.4</td>
</tr>
<tr>
<td>2S</td>
<td>1.4</td>
<td>1.3</td>
</tr>
<tr>
<td>3N</td>
<td>0.9</td>
<td>0.7</td>
</tr>
<tr>
<td>3S</td>
<td>0.9</td>
<td>0.8</td>
</tr>
</tbody>
</table>

\[ S_{R_{\text{max}}} = 2.4 \quad 0.5 \]

Table 5.4 – Summary of maximum, minimum, and stress ranges (in ksi) for floorbeam strain gages for the various test truck lane positions

*Note: Lane 1 is the inside lane, lane 3 is the outside lane*
5.3.4 Hangers

Strain gages were installed on several hanger plates. At each panel point, the tie girders are hung from the arches above via wire rope suspenders. The connection from the suspender end block and the girder consists of two plates which straddle the tie girder and end block.

Strain gages were placed on the outer of the two hanger plates. At panel point 14, the outer east hanger plate was instrumented on both sides (CH_5 and CH_6) to resolve axial and bending components. A single gage was placed on the outer surface of the outer west hanger plate at panel point 14 (CH_4). Finally, a single gage was installed on the outer east hanger plate at panel point 12 (CH_13). Figure 5.21 contains a stress time-history plot for the back-to-back gages on the east hanger plate at panel point 14 for the north bound crawl test in lane 3 (adjacent to the tie girder). Also shown in the plot is the stress history at CH_7, which represents the stress at the termination of the web longitudinal stiffener at panel point 14, to indicate the location of the test truck.

It can be seen from the plot that the stress in the hanger plate is low, with a maximum stress of 0.25 ksi. Furthermore, the hanger plate is put into maximum tension when the test truck is directly over panel point 14. The stress in the plate is predominantly the result of axial loading, however, there is a measurable bending component of approximately 0.04 ksi, or 16% of the total stress.

![Figure 5.21 – Stress time-history in east hanger plate at PP 14 during north bound crawl test in lane 3 (i.e. adjacent to tie girder)](image111x174 to 501x444)
A summary of the measured stresses in the instrumented hangers is presented in Table 5.5. It can be seen from the table that as the truck moves from lane 3N to 3S (east to west), the peak stress in the east hangers (CH_5, CH_6 and CH_13) decreases, while the peak stress in the west hanger (CH_4) increases. In all cases however, the stresses are low. Furthermore, if it is assumed that the full test truck load of 72 kips is carried by one floorbeam, when it is directly over that floorbeam and in the inside lane, the reaction at either end of the floorbeam will be close to 72 kips/2 = 36 kips. The width of the 1/2 inch hanger plate is 36 inches. Since there are two hanger plates, the stress would be 36 kips/(2x36 in.x1/2 in.) = 1 ksi. It can be seen that the actual measured stresses in the plate were a maximum of 0.20 ksi (equal to the average of CH_5 and CH_6, to eliminate bending effects). This discrepancy is the result of distribution of load to other floorbeams and the fact that the gage on the plate may be recording a stress that is lower than the nominal stress in the plate, due to the short length of the plate between the bolted connections.

<table>
<thead>
<tr>
<th>Truck in Lane</th>
<th>CH_5 (PP14 East)</th>
<th>CH_6 (PP14 East)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range Max Min</td>
<td>Range Max Min</td>
</tr>
<tr>
<td>3N</td>
<td>0.26 0.19 -0.07</td>
<td>0.38 0.33 -0.05</td>
</tr>
<tr>
<td>2N</td>
<td>0.21 0.17 -0.04</td>
<td>0.31 0.25 -0.07</td>
</tr>
<tr>
<td>1N</td>
<td>0.21 0.16 -0.05</td>
<td>0.31 0.24 -0.07</td>
</tr>
<tr>
<td>1S</td>
<td>0.15 0.12 -0.03</td>
<td>0.18 0.12 -0.06</td>
</tr>
<tr>
<td>2S</td>
<td>0.15 0.11 -0.05</td>
<td>0.18 0.15 -0.03</td>
</tr>
<tr>
<td>3S</td>
<td>0.24 0.13 -0.11</td>
<td>0.17 0.09 -0.09</td>
</tr>
</tbody>
</table>

\[ S_{R_{max}} = 0.26 \quad 0.38 \]

<table>
<thead>
<tr>
<th>Truck in Lane</th>
<th>CH_4 (PP14 West)</th>
<th>CH_13 (PP12 East)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range Max Min</td>
<td>Range Max Min</td>
</tr>
<tr>
<td>3N</td>
<td>0.17 0.11 -0.07</td>
<td>0.26 0.21 -0.06</td>
</tr>
<tr>
<td>2N</td>
<td>0.16 0.08 -0.08</td>
<td>0.17 0.11 -0.06</td>
</tr>
<tr>
<td>1N</td>
<td>0.21 0.13 -0.07</td>
<td>0.17 0.09 -0.08</td>
</tr>
<tr>
<td>1S</td>
<td>0.20 0.16 -0.05</td>
<td>0.12 0.09 -0.03</td>
</tr>
<tr>
<td>2S</td>
<td>0.26 0.23 -0.03</td>
<td>0.15 0.06 -0.10</td>
</tr>
<tr>
<td>3S</td>
<td>0.40 0.27 -0.13</td>
<td>0.20 0.09 -0.10</td>
</tr>
</tbody>
</table>

\[ S_{R_{max}} = 0.40 \quad 0.26 \]

Table 5.5 – Summary of peak measured stresses (in ksi) in the four hanger strain gages. CH_5 and CH_6 are back-to-back.

*Note: Lane 1 is the inside lane, lane 3 is the outside lane*
5.3.5 Dynamic Response

Controlled load tests were conducted in all lanes with the test truck traveling at normal traveling speed (35-50 mph for northbound tests and 30-40 mph for southbound tests) to assess the magnitude of dynamic amplification of stresses in the bridge. Figure 5.22 contains a stress time-history for the east tie girder gages, during a dynamic test with the truck traveling north in the middle lane. This figure can be compared with Figure 5.12 above, which shows an identical stress history for the crawl test. It can be seen from this figure that the peak stresses are similar to that of the crawl test. However, there is significantly more vibration in the girder caused by the passage of the test truck.

Figure 5.22 – Stress time-history for gages on the east tie girder 5’-10 1/2” north of PP 12 for dynamic test with north bound truck in middle lane.
A summary of the peak tension, compression and stress range for both the crawl and dynamic tests is shown in Table 5.6. Also given is the ratio between the two. The most observed dynamic amplification of stresses occurs in the hanger gages (CH_4, 5, 6, and 13), and the floorbeam gages (CH_1 and CH_3). These results are highlighted in Table 5.6. As seen in the table, the peak amplification of the stress range was 2.40 at CH_4. For the remaining gages, the amplification was not as significant, and generally less than 10%.

<table>
<thead>
<tr>
<th>Channel No.</th>
<th>Crawl Tests</th>
<th>Dynamic Tests</th>
<th>Ratio (dyn/ctrl)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SR (ksi)</td>
<td>Tens. (ksi)</td>
<td>Compr. (ksi)</td>
</tr>
<tr>
<td>Tie</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CH_11</td>
<td>1.59</td>
<td>0.83</td>
<td>-0.90</td>
</tr>
<tr>
<td>CH_12</td>
<td>1.66</td>
<td>1.12</td>
<td>-0.79</td>
</tr>
<tr>
<td>CH_14</td>
<td>1.41</td>
<td>0.82</td>
<td>-0.89</td>
</tr>
<tr>
<td>CH_15</td>
<td>1.32</td>
<td>0.80</td>
<td>-0.74</td>
</tr>
<tr>
<td>CH_16</td>
<td>1.50</td>
<td>1.05</td>
<td>-0.86</td>
</tr>
<tr>
<td>CH_17</td>
<td>1.45</td>
<td>1.02</td>
<td>-0.81</td>
</tr>
<tr>
<td>Floor beam</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CH_1</td>
<td>2.42</td>
<td>2.15</td>
<td>-0.27</td>
</tr>
<tr>
<td>CH_3</td>
<td>0.50</td>
<td>0.11</td>
<td>-0.44</td>
</tr>
<tr>
<td>Hanger</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CH_4</td>
<td>0.40</td>
<td>0.27</td>
<td>-0.13</td>
</tr>
<tr>
<td>CH_5</td>
<td>0.26</td>
<td>0.19</td>
<td>-0.11</td>
</tr>
<tr>
<td>CH_6</td>
<td>0.38</td>
<td>0.33</td>
<td>-0.09</td>
</tr>
<tr>
<td>CH_13</td>
<td>0.26</td>
<td>0.21</td>
<td>-0.10</td>
</tr>
<tr>
<td>Longit. Stiff</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CH_7</td>
<td>1.20</td>
<td>0.98</td>
<td>-0.38</td>
</tr>
<tr>
<td>CH_9</td>
<td>0.66</td>
<td>0.56</td>
<td>-0.17</td>
</tr>
<tr>
<td>Back Up Bar</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CH_18</td>
<td>1.46</td>
<td>0.99</td>
<td>-0.77</td>
</tr>
<tr>
<td>CH_19</td>
<td>1.68</td>
<td>1.16</td>
<td>-0.88</td>
</tr>
<tr>
<td>CH_23</td>
<td>1.23</td>
<td>0.93</td>
<td>-0.54</td>
</tr>
<tr>
<td>CH_26</td>
<td>0.13</td>
<td>0.09</td>
<td>-0.06</td>
</tr>
<tr>
<td>CH_27</td>
<td>0.13</td>
<td>0.10</td>
<td>-0.07</td>
</tr>
<tr>
<td>Diaph. Stiff</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CH_20</td>
<td>0.42</td>
<td>0.33</td>
<td>-0.18</td>
</tr>
<tr>
<td>CH_21</td>
<td>1.65</td>
<td>1.33</td>
<td>-0.73</td>
</tr>
<tr>
<td>CH_24</td>
<td>0.58</td>
<td>0.41</td>
<td>-0.31</td>
</tr>
<tr>
<td>CH_25</td>
<td>2.11</td>
<td>1.57</td>
<td>-1.04</td>
</tr>
<tr>
<td>Web Gap</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CH_10</td>
<td>0.61</td>
<td>0.21</td>
<td>-0.46</td>
</tr>
<tr>
<td>Access Hole</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CH_22</td>
<td>0.16</td>
<td>0.11</td>
<td>-0.09</td>
</tr>
</tbody>
</table>

Table 5.6 – Summary of peak stress-range, tension stress, and compression stress for the crawl and dynamic tests.
Also shown are the stress ratios (= Dynamic Stress/Crawl Stress)
5.4 Long-term Monitoring

The long-term monitoring of the tied arch was conducted from March 24 to April 15, 2004, for a total of 21.8 days of long-term data. Based on a review of the controlled load test data, 21 of the 25 channels used during the controlled load tests were selected for long-term monitoring. Stress time-history data were recorded in all 21 channels when a predefined trigger value of 2 ksi was exceeded in either channel CH_16 (bottom of east tie, for northbound traffic), or CH_12 (bottom of west tie for southbound traffic). For northbound traffic, data were recorded 14 seconds before and 8 second following trigger event. For southbound traffic, the pre and post trigger times were interchanged. Stress-range histograms were recorded continuously.

In addition, wind speed was constantly monitored during the long-term monitoring period. When the wind speed on the bridge exceeded 40 mph, data for all channels were recorded for 4 seconds before and 10 minutes subsequent to the trigger event.

5.4.1 Results of Long-term Monitoring

In this section, results of the fatigue evaluation of the tied arch bridge are presented. The rainflow cycle counting method was used to develop the stress-range histograms. Each detail type where monitoring was performed is analyzed and presented separately below. An in-depth discussion of the methodology used for the fatigue evaluation can be found in Appendix B.

5.4.2 Tie Girders

5.4.2.1 Global Tie Histogram

Figure 5.23 contains stress-range histograms for the six strain gages located on the top and bottom flanges of the east and west tie girders. Gages CH_11 and CH_12 are on the top and bottom, respectively, of the west girder. Gages CH_14 and CH_15 are on the top flange of the east girder and gages CH_16 and CH_17 are on the bottom flange.

There is some discrepancy between the two gages on the top and on the bottom. However, overall the stress range histograms are fairly consistent between the six locations.

These instrumented locations are classified as category C, because of the transverse welds of the interior diaphragms to the flange. All cycles less than 2.5 ksi were discarded, as indicated in Figure 5.23. Though these details are located inside the girder and the measured stresses are on the outside of the girder, the difference between the two should be minimal due to the large depth of the girder with respect to the flange thickness. Table 5.7 contains the estimates of the remaining life for these locations. The effective stress ranges are consistent. Furthermore, the CAFL was never exceeded during the monitoring period.

For all the gages, it has been determined that these details have infinite life remaining. Since these gages were located in an area of the tie determined to have the highest stresses by calculation, the infinite life assessment can be extended to the entire tie girder.
Figure 5.23 – Stress-range histogram for strain gages located on the top and bottom flanges of the east and west tie girders

Table 5.7 – Summary of life calculations for transverse connections to the tie girders

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>$S_{R_{max}}$ (ksi)</th>
<th>Cycles &gt; CAFL</th>
<th>$S_{Reff}$ (ksi)</th>
<th>Cycles/day</th>
<th>Remaining Life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_11</td>
<td>5.0</td>
<td>0</td>
<td>2.9</td>
<td>17</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_12</td>
<td>5.0</td>
<td>0</td>
<td>2.9</td>
<td>31</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_14</td>
<td>5.5</td>
<td>0</td>
<td>3.0</td>
<td>10</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_15</td>
<td>5.5</td>
<td>0</td>
<td>3.0</td>
<td>4</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_16</td>
<td>6.0</td>
<td>0</td>
<td>2.9</td>
<td>18</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_17</td>
<td>6.0</td>
<td>0</td>
<td>2.9</td>
<td>20</td>
<td>infinite</td>
</tr>
</tbody>
</table>
5.4.2.2 Diaphragm Stiffeners

Figure 5.24 contains the stress-range histograms for the strain gages mounted to the diaphragm stiffeners. Note that two locations were instrumented. Gages CH_20 and CH_21 were located at panel point 13 while gages CH_24 and CH_25 were located at panel point 11. Gages CH_20 and CH_24 were mounted to the stiffeners, while gages CH_21 and CH_25 were mounted to the tie girder web at the termination of the stiffener/web weld. The stresses recorded in gages CH_20 and CH_24 were minimal. Infinite life is expected at these locations. The stress-range histograms for gages CH_21 and CH_25 are presented in Figure 5.24. It can be seen that the number and magnitude of cycles at CH_25 was much higher.

Table 5.8 contains the estimates for the remaining fatigue life for these details, which can be assumed to be representative of all such details on this bridge. These details were assumed to be classified as category E with a CAFL of 4.5 ksi. Stress cycles less than 1.0 ksi were discarded, as indicated in Figure 5.24. It can be seen that although the effective stress range was below the CAFL for both locations, the CAFL was exceeded with a frequency of more than 1/10,000. Therefore, the detail is deemed to have finite life. The finite life branch of the S-N curve (the sloped portion) is extended downwards to estimate the fatigue life. However, based on the rate of stress cycles, the remaining life for these details is more than 150 years, effectively infinite.

![Stress-range histogram for strain gages located on the girder web at the diaphragm/web weld toe](image-url)
<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>(S_{\text{Rmax}}) (ksi)</th>
<th>Cycles &gt; CAFL</th>
<th>(S_{\text{Roff}}) (ksi)</th>
<th>Cycles/day</th>
<th>Remaining Life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_21</td>
<td>6.0</td>
<td>7</td>
<td>0.02</td>
<td>1.6</td>
<td>1,709 over 100</td>
</tr>
<tr>
<td>CH_25</td>
<td>8.0</td>
<td>41</td>
<td>0.07</td>
<td>1.9</td>
<td>2,570 over 100</td>
</tr>
</tbody>
</table>

Table 5.8 – Summary of life calculations for diaphragm stiffener details

5.4.2.3 Backing Bars

Backing bars were used to make the full penetration welds at each corner of the tie girder. The backing bars are ½ inch wide x ¾ inch high. These details can be classified as category C short attachments for longitudinal stresses since the backing bars pass through the diaphragms. To determine the potential for fatigue cracking at these details, stress-range histograms were developed for a few locations.

The stress-range histograms for the backing bar details are presented in Figure 5.25. Three locations were instrumented: at panel point 11 (CH_26 and CH_27), panel point 12 (CH_18 and CH_19), and a single gage at panel point 13 (CH_23). Gages CH_18, CH_23, and CH_26 are located on the backing bar while gages CH_19 and CH_27 are located on the bottom flange adjacent to the backing bar. It can be seen that the histograms for these gages are fairly consistent.

As a first approach, the remaining fatigue life was evaluated using a simple nominal stress approach assuming category C represents the fatigue category for the detail. For this approach, all cycles less than 2.5 ksi were discarded for this analysis, as indicated in Figure 5.25. Table 5.9 contains the estimates of the remaining life for these locations. The effective stress ranges are consistent. Furthermore, the CAFL was never exceeded during the monitoring period. For all the gages, it has been determined that these details have infinite life remaining, assuming category C.

However, as shown on the design drawings, the diaphragms are cut out around the continuous backing bars. It should be noted however, that there is a potential for fatigue cracks to grow out of the throat of the welds to the diaphragm plate and into the flange. This is dependant on the magnitude of the stresses that are passing out of the backing bar through the diaphragm welds. The potential for this to occur was evaluated by building a finite element model of the detail. The model utilized 20-node solid elements and represented a corner portion of the box (see Figure 5.26.)

Loads were applied assuming uniform stress flow through the flange and web and no global bending was included. This approach is reasonable and conservative as there is a relatively small strain gradient through the depth of the tie girder over this small region due to global bending of the tie girder. As shown in Figure 5.27, the longitudinal stress flow through the welds is relatively small and the continuous backing bar does not shed much load through the weld. It is important to note that the actual magnitudes of the stresses in the analysis are not important. Rather, the relative magnitudes between the stresses in the flange and weld are of interest. The stress concentration at the weld toe on the flange and backing are apparent in the Figure. (There is always a stress concentration at a weld toe and this effect is included in the associated fatigue category.) Since the stress flow carried by the weld is not greater than anticipated, the assignment of
category C is reasonable and cracking from within the throat is believed to be very unlikely. This detail can be thought of as being similar to a fully welded transverse stiffener connection.

Figure 5.25 – Stress-range histogram for strain gages located on the backing bars

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>$S_{\text{Rmax}}$ (ksi)</th>
<th>Cycles $&gt;$ CAFL</th>
<th>$S_{\text{Reff}}$ (ksi)</th>
<th>Cycles/day</th>
<th>Remaining Life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_18</td>
<td>5.5</td>
<td>0</td>
<td>2.9</td>
<td>14</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_19</td>
<td>5.0</td>
<td>0</td>
<td>2.9</td>
<td>40</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_23</td>
<td>4.5</td>
<td>0</td>
<td>3.0</td>
<td>2</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_26</td>
<td>6.0</td>
<td>0</td>
<td>2.9</td>
<td>11</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_27</td>
<td>5.5</td>
<td>0</td>
<td>2.9</td>
<td>23</td>
<td>infinite</td>
</tr>
</tbody>
</table>

Table 5.9 – Summary of life calculations for backing bar details
Figure 5.26 - View of solid model used to establish stress flow through weld at backing bar-to-diaphragm connection

Figure 5.27 - View of longitudinal stress contours at backing bar-to-diaphragm connection
5.4.2.4 Longitudinal Stiffeners

Figure 5.28 contains a stress-range histogram for the two longitudinal stiffener details that were instrumented. Gage CH_7 was located at the termination of the longitudinal stiffener, and gage CH_9 was located at the intersection with the diaphragm.

These instrumented locations are classified as category E. All cycles less than 1.0 ksi were discarded, as indicated in Figure 5.28. Table 5.10 contains the estimates of the remaining life for these locations. It can be seen that the effective stress range is less than the CAFL and that the CAFL was never exceeded. Therefore, these details have infinite life remaining.

![Stress-range histogram for strain gages located at longitudinal stiffener details (CH_7 at termination, and CH_9 at intersection with diaphragm)](image)

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>$S_{\text{Rmax}}$ (ksi)</th>
<th>Cycles $&gt;$ CAFL</th>
<th>$S_{\text{Reff}}$ (ksi)</th>
<th>Cycles/day</th>
<th>Remaining Life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_7</td>
<td>4.5</td>
<td>0</td>
<td>1.4</td>
<td>827</td>
<td>infinite</td>
</tr>
<tr>
<td>CH_9</td>
<td>2.5</td>
<td>0</td>
<td>1.3</td>
<td>27</td>
<td>infinite</td>
</tr>
</tbody>
</table>

Table 5.10 – Summary of life calculations for longitudinal stiffener details

5.4.3 Floorbeam

Finally, a stress-range histogram for the gusset detail on the floorbeam is shown in Figure 5.29. Channel CH_1 was located at the termination of the gusset, on the opposite side of the transverse connection plate.
This instrumented detail is classified as category E. Therefore, all cycles less than 1.0 ksi were discarded, as indicated in Figure 5.29. Table 5.11 contains the estimates of the remaining life for this location. It can be seen that the effective stress range is less than the CAFL. There were four observed cycles with a stress range greater than the CAFL, however the frequency of occurrence of these cycles was less than 1/10,000. Therefore, this detail is deemed to have infinite life remaining.

Figure 5.29 – Stress-range histogram for strain gages located at termination of gusset plate on floorbeam web

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>$S_{R_{\text{max}}}$ (ksi)</th>
<th>Cycles $&gt; CAFL$</th>
<th>$S_{\text{eff}}$ (ksi)</th>
<th>Cycles/day</th>
<th>Remaining Life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH_1</td>
<td>5.0</td>
<td>4</td>
<td>0.006</td>
<td>1.6</td>
<td>infinite</td>
</tr>
</tbody>
</table>

Table 5.11 – Summary of life calculations for floorbeam gusset detail
5.4.4 Displacements of the Bridge

Displacements of the bridge were determined using the acceleration time-history data recorded based on the wind speed triggered data. As noted above, there were three channels of accelerometer data, all located at panel point 11. The three channels consisted of a vertical accelerometer at each tie girder, and a lateral accelerometer on the east tie girder. The acceleration data from each channel were first high pass filtered above 0.3 Hz. This was done to eliminate low frequency drift that produces erroneous results after numerical integration. Natural frequencies less than 0.3 Hz are not expected on a bridge such as this. The vertical response was calculated by taking half the sum of the two vertical time-histories. The torsional response was calculated by taking half the difference of the two vertical time-histories.

The velocity time-histories were then determined by integrating the acceleration histories utilizing the trapezoidal rule. Following this, the data were high-pass filtered and integrated once again to yield the displacement time-histories. After a final high pass filtering, the data appear as presented in Figure 5.30. In the lower part of the figure, a zoom-in of part of the data is shown. Note that the torsional displacements are given in terms of inches, and represents the vertical displacement of one tie girder (the other tie girder would have the same displacement in the opposite direction). In order to determine rotations, the torsional displacement can be divided by half the bridge width, equal to 55 feet, or 660 inches.

It can be seen from the figure that the calculated displacements are small, less than 1/32 inch in the lateral direction, 1/8 inch torsionally (less than 0.01 degrees), and 3/8 inch vertically. The peak displacements are summarized in Table 5.12. The largest displacements were in the vertical direction, at the lowest modal frequency of 0.4 Hz.

It should be noted however, that these estimates do not include the low frequency “static” vertical displacement that would be caused by passage of vehicles or lateral displacement due to a steady wind, since the vibration with a frequency of less than 0.3 Hz were filtered out.

<table>
<thead>
<tr>
<th>Direction</th>
<th>Minimum Displacement (in)</th>
<th>Maximum Displacement (in)</th>
<th>Standard Deviation (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral</td>
<td>-0.019</td>
<td>0.023</td>
<td>3.79x10^{-3}</td>
</tr>
<tr>
<td>Torsional</td>
<td>-0.110</td>
<td>0.090</td>
<td>1.23x10^{-2}</td>
</tr>
<tr>
<td>Vertical</td>
<td>-0.348</td>
<td>0.356</td>
<td>5.63x10^{-2}</td>
</tr>
</tbody>
</table>

Table 5.12 – Summary of peak displacements of the bridge at panel point 11 determined by numerically integrating measured acceleration histories during high wind events
Figure 5.30 – Displacement time-history of panel point 11 in the lateral, torsional, and vertical directions
5.4.5 Results of Video Monitoring

As described above, a video camera was installed on the tied arch span. When a trigger was detected by the logger, the logger triggered the camera and video was recorded for a predefined period of time, in this case equal to 5 seconds. The video clips which were collected during the monitoring period were extremely useful in determining the configuration of the vehicles causing the triggers. It was found that in most cases the triggered stress was caused by multiple trucks on the bridge at one time. However, there were also single vehicles which also produced high stresses in the bridge, such as mobile cranes and more interestingly, permitted “superloads.” Six crossings of superloads were observed during the monitoring period. All trucks were traveling in the northbound direction in the middle lane. PennDOT provided the permit information for each of the vehicles. Each vehicle consisted of a 4 axle tractor and a 15 axle articulated trailer, with a maximum gross vehicle weight of approximately 194 tons. Two of the permitted superloads also had a second “pusher” truck, identical to the lead truck at the rear. Figure 5.31 contains a series of photographs of one of the superloads (with a pusher truck) crossing the bridge as recorded by the video camera. Table 5.13 contains a summary of the superloads that crossed the bridge during the monitoring period.

<table>
<thead>
<tr>
<th>Crossing Number</th>
<th>Crossing Time</th>
<th>Direction</th>
<th>Lane</th>
<th>Total Weight(^1) (lb)</th>
<th>Total Length (ft)</th>
<th># axles</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3/24/2004 13:05</td>
<td>NB</td>
<td>2</td>
<td>344,000</td>
<td>138</td>
<td>19</td>
</tr>
<tr>
<td>2</td>
<td>3/25/2004 8:36</td>
<td>NB</td>
<td>2</td>
<td>388,200</td>
<td>175</td>
<td>19</td>
</tr>
<tr>
<td>3</td>
<td>3/29/2004 11:48</td>
<td>NB</td>
<td>2</td>
<td>372,000</td>
<td>156</td>
<td>19</td>
</tr>
<tr>
<td>4</td>
<td>3/29/2004 14:01</td>
<td>NB</td>
<td>2</td>
<td>378,000</td>
<td>178</td>
<td>19</td>
</tr>
<tr>
<td>5</td>
<td>4/1/2004 12:13</td>
<td>NB</td>
<td>2</td>
<td>344,000</td>
<td>138</td>
<td>19</td>
</tr>
<tr>
<td>6</td>
<td>4/2/2004 11:27</td>
<td>NB</td>
<td>2</td>
<td>388,200</td>
<td>175</td>
<td>19</td>
</tr>
</tbody>
</table>

1Total weight obtained from permit data

Table 5.13 – Summary of “superload” crossings recorded by the triggered video
Table 5.14 presents the measured stress range at selected gages during the crossing of the six superloads. Gages CH_12 and CH_17 are on the east and west tie girder, respectively. Gage CH_1 is on the floorbeam at PP 14. Finally, gages CH_4 and CH_5 are on the east and west hangers at PP14, respectively. Since the ties respond to global load position rather than local load position, it can be seen that the stress ranges are significantly higher than those recorded during the load tests (= 1.7 ksi for CH_12 and 1.5 ksi for CH_17). The stresses recorded during the superload crossings were more than double those measured during the load test. However, since the hangers and floorbeams respond to local load position, the difference is not as great. A stress range of 2.4 ksi was recorded at CH_1 during the load tests. Stress ranges of 0.3 and 0.4 ksi were recorded at gages CH_4 and CH_5 during the load tests. The hanger stresses are not significantly different during the superload crossings.

<table>
<thead>
<tr>
<th>Superload No.</th>
<th>East Tie CH_12 $S_R$ (ksi)</th>
<th>West Tie CH_17 $S_R$ (ksi)</th>
<th>Floorbeam CH_1 $S_R$ (ksi)</th>
<th>East Hanger CH_4 $S_R$ (ksi)</th>
<th>West Hanger CH_5 $S_R$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.6</td>
<td>5.4</td>
<td>3.9</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>2</td>
<td>4.3</td>
<td>5.2</td>
<td>3.9</td>
<td>0.5</td>
<td>0.6</td>
</tr>
<tr>
<td>3</td>
<td>4.7</td>
<td>5.8</td>
<td>3.4</td>
<td>0.5</td>
<td>0.6</td>
</tr>
<tr>
<td>4</td>
<td>4.0</td>
<td>4.7</td>
<td>3.7</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>5</td>
<td>4.9</td>
<td>5.6</td>
<td>4.5</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>6</td>
<td>5.0</td>
<td>5.8</td>
<td>4.4</td>
<td>0.5</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Table 5.14 – Summary of measured stress ranges at selected strain gages during crossing of the six superloads.
Figure 5.32 contains a stress-time history plot for the stress in the east tie girder (channels CH_14, 15, 16 and 17). Also shown is the stress-history for the stress at the end of the gusset plate on the floorbeam (CH_1). It can be seen, that the individual axle loads cannot be seen in the stress history of the tie girder. This is because the bending and axial stress in the tie girder is not sensitive to the number and spacing of individual axles. The magnitude of stress is more a function of the gross vehicle weight. Note that the peak stress in the bottom flange of the girder (CH_16 and 17) is on the order of 3.5 ksi. Comparing Figure 5.32 to Figure 5.12, the stress due to the super load is approximately 3.5 ksi/0.9 = 3.9 times larger than the stress produced by the test truck. This ratio is less than the ratio of the vehicle weights (= 388 kips/72.1 = 5.4). This discrepancy can be attributed to the fact that the peak stress in the tie is reduced due to the length of the load (i.e. it is not 388 kips applied at a point). Furthermore, the actual weight of the super load is unknown, but most likely less than the permitted weight.

The effect of these superloads on the remaining fatigue life is included in the life calculations presented above, since stress cycles caused by these vehicles was measured and included in the stress-range histogram. However, due to the low frequency of occurrence of these vehicles, the effect is on the fatigue life is minimal.

Figure 5.32 – Stress time-history for the east tie girder (CH_14, 15, 16, 17) and the floorbeam (CH_1) for the during the passage of superload number 6
5.4.6 Dynamic Properties of the Tied Arch

Using the continuous time-history data collected during the wind triggering, some of the dynamic properties of the bridge can be investigated. Specifically, the resonant frequencies of the bridge and the damping ratios can be estimated. If the bridge is excited at a frequency equal to a resonant frequency, the response will be much greater than that at a non-resonant frequency.

The damping ratio defines how quickly the bridge returns to an undisturbed state in free vibration after being dynamically excited. It is related to the amount of energy dissipation inherent in the structure. From a fatigue point of view, this is important because a structure with a low damping ratio (such as a light pole) can be subjected to many more cycles due to a single loading event (such as a wind gust) because the free vibration lasts longer than a structure with a higher damping ratio (such as a riveted bridge).

It should be noted that the bridge has many resonant frequencies. Each frequency represents a “mode of vibration” of the bridge. Each mode has a damping ratio associated with it. The resonant frequencies and damping ratios are properties of the bridge, such as stiffness or mass.

5.4.6.1 Resonant Frequencies

In order to determine the resonant frequencies, the Fast Fourier Transform (FFT) algorithm was used. This algorithm decomposes a time-varying signal into a suite of sine and cosine waves of different frequencies and amplitudes. When all of these sine and cosine waves are added together, the original time-history is obtained. Plotted in Figure 5.33 are the FFT’s of the lateral, torsional and vertical acceleration records recorded during the wind triggering (this is the same time history used in the displacement calculations above.) The horizontal axis is the frequency of the signal, while the vertical axis represents the amplitude of the sine wave at that frequency. Each peak on the plot represents a resonant frequency, and a separate “mode of vibration” of the structure. The lowest mode of vibration is a vertical mode with a frequency of 0.41 Hz. Table 5.15 contains a summary of the modes identified from the FFT plot. Note that in some cases, modes are coupled, that is, a mode has a peak in two directions of motion, such as lateral and torsional.
Figure 5.33 – FFT of the acceleration response of the bridge at panel point 11, in the lateral, torsional, and vertical directions
5.4.6.2 Damping Ratios

The damping ratios for selected modes were determined using two methods. Namely, the half-power bandwidth (HPBW) method, and the random decrement (RD) method. The HPBW method utilizes results in the frequency domain (the FFT), while the RD method uses the time-history data. Table 5.15 contains the damping ratios, $\xi$, for the various modes identified above. It can be seen that there is good agreement between the damping ratios calculated by the two methods. The highest damping ratio was calculated for the mode “Lat1”, equal to 3%. The lowest damping exists in mode “Tor3” equal to 0.3%.

It is important to note that during the monitoring period, the largest stresses were caused by the primary loading of vehicles. The stress cycles caused by free vibration after removal of the load were minimal, and would not add significantly to the cumulative fatigue damage.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Freq (Hz)</th>
<th>$\xi$ RD</th>
<th>$\xi$ HPBW</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lat1</td>
<td>0.71</td>
<td>2.89%</td>
<td>2.98%</td>
</tr>
<tr>
<td>LatTor2</td>
<td>0.95</td>
<td>0.58%</td>
<td>0.74%</td>
</tr>
<tr>
<td>Lat3</td>
<td>0.99</td>
<td>1.15%</td>
<td>1.20%</td>
</tr>
<tr>
<td>LatTor4</td>
<td>1.29</td>
<td>0.45%</td>
<td>0.62%</td>
</tr>
<tr>
<td>LatVer5</td>
<td>1.97</td>
<td>0.67%</td>
<td>0.53%</td>
</tr>
<tr>
<td>LatVer6</td>
<td>2.42</td>
<td>0.70%</td>
<td>0.41%</td>
</tr>
<tr>
<td>Tor3</td>
<td>1.71</td>
<td>0.29%</td>
<td>0.35%</td>
</tr>
<tr>
<td>Ver1</td>
<td>0.41</td>
<td>2.18%</td>
<td>2.18%</td>
</tr>
<tr>
<td>Ver2</td>
<td>1.03</td>
<td>0.68%</td>
<td>0.93%</td>
</tr>
</tbody>
</table>

Table 5.15 – Identified modes and damping ratios, $\xi$ determined by the RD and HPBW methods
6.0 Summary and Conclusions

The following section provides a brief summary of the results of the field instrumentation, testing, and monitoring conducted on the various spans of the Neville Island Bridge. In addition, some discussion pertaining to required retrofits is included.

6.1 Ramp J

Based on the results of the field instrumentation, monitoring, and destructive testing, the following conclusions are made:

- **Tab Plate Details and General Girder Response**
  - Live load effective stress ranges are low at the tab plate weld toe and the CAFL is only occasionally exceeded.
  - The girders and transverse floorbeams behave compositely under service loads.
  - Assuming no increase in traffic pattern, remaining fatigue lives of over 100 years were calculated at instrumented tab plates based on the month long monitoring of ramp J.
  - No fatigue or fabrication cracks were observed in cores taken from Ramp J and weld quality was typically good.
  - Based on examination of cores taken from the same type of details on Ramp H two years ago, it was found that fabrication cracks, present for nearly 30 years, did not grow from fatigue and remained benign.
  - Based on the above, there is no need to retrofit the tab plate details. Since Ramp J was determined by analysis to have some of the highest stress ranges, these results can be reasonably extended to the other curved girder ramps with similar details. However, if it is felt that some form of retrofit is needed, a suggested detail is included in Appendix D. However, this retrofit is the most that should be considered to improve the fatigue resistance of the detail.

- **Longitudinal Stiffener Details**
  - Based on the results of the long-term monitoring, there does not appear to be a need to retrofit any of the longitudinal stiffener termination for fatigue. The longitudinal stiffeners on Ramp J were selected for instrumentation because analysis of Ramp J indicated that these had the highest stress ranges. Hence, these results can be reasonably extended to the other curved girder ramps with similar details.
  - Although no retrofits are required for the fatigue limit state, the longitudinal stiffener terminations which intersect the vertical stiffeners and transverse connection plates which have less than a 1/4" gap should be retrofit to prevent constraint induced fracture. A suggested retrofit detail is provided in Appendix D. It should be noted that other acceptable retrofit details exist and should be considered if thought to more economical.
6.2 **Ramp H**
The results of filed instrumentation and long-term monitoring of ramp H are summarized below.

- **Girder Web at Transverse Connection Plate Detail**
  - The result of the long-term monitoring suggests that no fatigue cracking due to out-of-plane distortion is expected at the instrumented details.
  - Based on the one month monitoring period, an infinite fatigue life was estimated at the two locations instrumented.

6.3 **Span 25 and 26**
The following summarized the results of the field instrumentation and long-term monitoring of spans 25 and 26.

- **K Cracking**
  - The cause for the observed K-cracking is believed to be related to incompatibility between the concrete deck and the floorbeams. The incompatibility results in a transverse relative displacement between the end of the floorbeam top flange and the bottom of the slab. This behavior was described in detail in Chapter 4.
  - The stress-range histograms developed at these details obtained during the long-term monitoring confirm that cracking would be expected at this connection. Hence, the observed cracking is consistent with the measured data.
  - Since the cracks are driven by displacement, it is likely they will continue to grow with time. However, there are several locations where cracks have been identified during routine inspection which have not grown for several years. Rather than proceed with an expensive retrofit at all locations, it is suggested that only those connection possessing cracks of at least 1 inch be retrofitted at this time. However, all cracks less than 1 inch which have exhibited substantial recent growth should also be retrofitted. The recommended detail is similar to the standard approach used to prevent out-of-plane distortion cracking at details where the transverse connection plates are not welded to the flange.
  - The suggested retrofit strategy is believed to be the most economical approach. The only sure way to prevent further future cracking is to directly attach the slab to the top of the floorbeam. This would be a very expensive retrofit and potentially unnecessary (*It is thought to be potentially unnecessary since many of the cracks have not grown for several years*). Since cracking of other connections is not going to lead to any serious problems (*i.e., a girder will not fracture*) and the cracking process is very slow, it seems that the suggested approach is the most reasonable. If additional cracks develop, there is sufficient time to repair them well before any serious problems develop.
• **Floorbeam Cope Cracking**
  o The results of the field instrumentation confirm that additional cracking can be expected at the existing floorbeam copes. The copes exist at each end of the floorbeams at the connection of the floorbeam to the transverse connection plates.
  o The recommended retrofit is identical to that previously used at other cracked floorbeams on the bridge.

• **Longitudinal Stiffener Details**
  o Based on the results of the long-term monitoring, there does not appear to be a need to retrofit any of the longitudinal stiffener termination for fatigue. The longitudinal stiffeners on Span 26 were selected for instrumentation because analysis indicated that these had some of the highest calculated stress ranges of the mainline bridges. Hence, these results can be reasonably extended to the other mainline plate girder spans with similar details.
  o Although no retrofits are required for the fatigue limit state, the longitudinal stiffener terminations which intersect the vertical stiffeners and connection plates which have less than a ¼” gap should be retrofitted to prevent constraint induced fracture. A suggested retrofit detail is provided in Appendix D. It should be noted that other acceptable retrofit details exist and should be considered if thought to be more economical.

6.4 **Tied Arch**

Field instrumentation and monitoring of the tied arch span was carried out and a fatigue evaluation of critical details has been completed. In addition, three cores were removed from the web of the tie girder for destructive evaluation to examine weld quality at intersecting weld details potentially susceptible to fracture and fatigue. The results are summarized below:

• Assuming no increase in traffic pattern, the fatigue life estimate at all instrumented details is over 100 years and most have infinite life. Hence, no cracking due to load induced fatigue is expected.

• The destructive evaluation of the three cores taken from the tied arch is summarized in Appendix C. A summary of the results of this investigation indicate the following:
  o There was no evidence of fatigue cracking in any of the cores. This observation is consistent with the field measured stress ranges. Hence, no future cracking is expected.
  o The weld quality was consistent with this type of detail and the period of fabrication. Examination revealed no unusual defects.
  o A fracture mechanics analysis indicates that the fracture of these intersecting weld details appears to be very unlikely that no further retrofits are required.
Appendix A

Instrumentation Plans
PARTIAL PLAN - RAMP H
SCALE: 1/32" = 1'-0"

SECTION A-A
SCALE: 1/4" = 1'-0"

STIFFENER 6 1/2" F.B.
KNEE BRACE 6 9/16" F.B.
CH.36 (INSIDE)
CH.37 (OUTSIDE IN.L.NE)
WITH CH.36)
CH.38 (L.V.D.T.)
MEASURING DISP. OF STIFF.
RELATIVE TO GIRDER FLANGE

DETAIl B
SCALE: 1 1/2" = 1'-0"

GIRDER G2

KNEE BRACE 6 9/16"
TRANSVERSE STIFFENER 6 1/2"

DETAIl C
SCALE: 1 1/2" = 1'-0"
Appendix B

Development of Stress-Range Histograms used to Calculate Fatigue Damage
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., $S_{\text{reff}} = \left[\Sigma \alpha_i S_{ri}^3\right]^{1/3}$).

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 (18µε). 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 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 ¼ 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, 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.
References:

Appendix C

Evaluation of Ramp-J and Tied-arch Core Samples
C1. **Ramp J Core Samples**

Two cores were taken from the bottom flange of ramp J at the locations where the transverse tab plate is welded to the edge of the flange. This detail and the location of a typical core are shown in Figure C.1. A removed core is shown in Figure C.2.
The two cores were sectioned into smaller pieces, sanded, polished and etched to establish the quality of the welds and determine if any fatigue cracking has occurred. One piece, which is typical of all specimens, is shown in detail in Figure C.3. As can be seen, small lack of fusion defects exist in the full-penetration weld used to attach the tab plate to the edge of the flange. Weld defects such as this are common. They are not detrimental to the fatigue performance of these details at the live load stress levels measured. Hence, none of the observed defects appear to have grown in fatigue as expected. (*Note that the field measurements in one of the tab plates demonstrated that transverse stresses are very low, less than 1.0 ksi*).

It is important to mention that this observation is consistent with the findings made during the examination of the two cores taken from Ramp H a few years ago. These two cores were taken when cracks were found during the removal of the same type of tab plates on Ramp H. The cores were examined at Lehigh University and were determined to be fabrication cracks, which did not exhibit any growth in fatigue.

More importantly, no evidence of weld toe cracking was observed at the termination of the longitudinal weld used on the flange edge in any of the cores recently removed and examined from ramp J. Again a finding consistent with the field measurements on ramp J which indicated that fatigue cracking at these details (category E) would not be expected to occur.
C1.1 Summary of Ramp-J Cores findings

In summary, the follow observations have been related to the transverse tie plates as a result of these investigations.

- Live load effective stress ranges are low at the weld toe and the CAFL is only occasionally exceeded.
- Remaining fatigue lives of over 100 years were calculated at instrumented tab plates based on the month long monitoring of ramp J.
- No fatigue or fabrication cracks were observed in cores taken from Ramp J and weld quality was typically good.
- Based on examination of cores taken from the same type of details on Ramp H two years ago, it was found that fabrication cracks, present for nearly 30 years, did not grow fatigue and remained benign.
C2. Tied-Arch Core Samples

Three cores were taken from the tied arch box girder. The three cores were sectioned into smaller pieces, sanded, polished and etched to establish the quality of the welds and to determine if any fatigue cracking has occurred. The first core was taken at panel point pp. 13. The second and the third cores were taken between panel point pp. 14 and pp. 15. Figure C.4 shows the locations at which the three cores were extracted.
Figure C.4 - location of the three cores extracted at the tied arch

All three cores (core #1, core #2 and core #3) had typical slag inclusion defects, which typically develop in the welding process. The size and the orientation of the defects were examined. It was concluded that these defects are not a cause for concern. In general the cores have good quality welds with no sign of crack propagation in either the welds or the web of the cores.

In core #2, a lack-of-fusion defect was also formed as a result of not removing the backup bar after the completion of the welding process. Core #2 (Figure C.5), which includes the backing bar showed no sign of fatigue crack propagation in either the HAZ
of the weld metal or in the web of the tie. However, because fatigue crack propagation and fracture have been known to occur in such a detail, a fatigue and fracture assessment was conducted (see below) to evaluate the integrity of the detail. Based on the assessment below, there appears to be no cause of concern from a fatigue and fracture standpoint.

Based on these findings, retrofits at any of the three details are not needed. Under the assumption that traffic patterns do not change drastically, there is no evidence to suggest that fatigue cracks should form at these locations. Furthermore, fracture of the weld metal in the backing bar is not expected to occur.

C3. Fatigue and Fracture Assessment of Backing bar Detail (Core # 2)

The core was taken at a detail where a backing-bar was used in the welding processes and was not removed upon the completion of welding. The lack-of-fusion at the detail (Figure C.6) creates a crack like condition which could be prone to propagation under fatigue and eventually fracture of the detail.
Figure C.5 - Core sample #2 taken for evaluation including the tie web, backing bar, and the diaphragm
Figure C.6 - 100 X magnification of the tip of the lack-of-fusion defect and a crack-like condition in the backing bar

- Diaphragm plate
- Weld metal of HAZ
- Backing bar
C3.1  Fatigue crack propagation

C3.1.1 HAZ of weld metal and web of tie girder

The lack of fusion defect in the backing bar could be idealized as an edge crack in which the change in the stress intensity factor is described in Equation C.1

\[ \Delta K = 1.12 \Delta \sigma \sqrt{\pi a} \]  

Equation C.1

Where, \( a \) = size of the defect = 0.23 inches

Defects and crack like conditions will not propagate if

\[ \Delta K \leq \Delta K_{\text{th}} = 3 \text{ksi} \sqrt{\text{in}} \]

Therefore, a measured value of \( \Delta \sigma < 3.15 \text{ ksi} \) would be required to assure that \( \Delta K \leq 3 \text{ ksi} \). Under such condition the lack-of-fusion defect would not propagate under fatigue in the HAZ or in the web of the tie girder.

The calculated \( \Delta \sigma = 3.15 \text{ ksi} \) was compared against the maximum absolute stress range on the longitudinal stiffener near the web of the tie (CH_9). At that location, the maximum absolute stress range measured during the long term monitoring is 2.0 ksi. Hence no fatigue crack propagation would be expected in the HAZ of the weld metal or in the web of the tie. This conclusion is confirmed by the fact that the bridge has been in service for approximately 30 years, and that the lack-of-fusion defect examined under the microscope and showed no evidence of fatigue crack propagation to date.

C3.2  Fracture assessment

C3.2.1 Web of the tie and HAZ of weld metal

Assessing the fracture potential of the web of the tie is of no concern since the lack-of-fusion defect did not propagate under fatigue and can not enter the web of the tie unless crack growth occurs at the lack-of-fusion defect and subsequently extend into the web.

Fracture in the HAZ of the weld metal can not develop if:

\[ K_{\text{max}} \leq K_{\text{IC}} \]

Where \( K_{\text{max}} \) can be conservatively estimated using Equation 1

\[ K_{\text{max}} = 1.12 \sigma_y \sqrt{\pi a} \]

Where

\( a \) = size of the defect = 0.23 inches
\( \sigma \) = maximum stress = yield point

The maximum stress intensity factor for the lack-of-fusion defect would depend on the residual stress state at the location of the defect. The magnitude of the residual
stress at the location where $Z = 0$ is maximum and could be assumed equal to the yield stress of the material (i.e. 50 ksi). An upper bound value of the yield stress ($\sigma_y$) could be estimated to be 60 ksi, which takes into account the combined dead load, live load, and residual stresses.

$$K_{max} = 1.12 \times 60 \sqrt{\pi a}$$

Where, $a$ = size of the defect = 0.23 inches

$$\therefore K_{max} = 1.12 \times 60 \sqrt{\pi \times 0.23}$$

$$\therefore K_{max} = 57.12 \text{ksi} \sqrt{\text{in}}$$

A reasonable assumption of the magnitude of the fracture toughness of the HAZ weld metal could be taken equal to the fracture toughness of the structural steel used for fabricating the web. The SMAW used in welding process have been known to exhibit a fracture toughness which is very comparable to that of the structural steel. Fisher et. al [1] conducted Charpy tests on core specimens taken from the I-79 Bridge and reported that the fracture toughness of the web of the tie girder was significantly high. A $K_{IC}$ value of 122 ksi $\sqrt{\text{in}}$ was reported when conducting the Charpy test at -100 °F. It is also a common practice that when the fracture toughness value of weld metal or structural steel is not know, then a $K_{IC}$ lower bound value of 100 ksi $\sqrt{\text{in}}$ could be estimated. In both cases, $K_{max} = 57.12 \text{ksi} \sqrt{\text{in}} \leq K_{IC} = 122 \text{ksi} \sqrt{\text{in}}$ or $100 \text{ksi} \sqrt{\text{in}}$, indicating that it is extremely unlikely that fracture would occur.

Reference:
Appendix D

Retrofit Plans
**NOTES:**

1. INSPECT AND GRIND INTERSECTIONS OF HOLE EDGES W/ FLANGE AND TAB P AND HOLE SURFACE AS PER HOAN RETROFIT DETAILS FOR DISTRICT 11-0
NOTES:
1. INSPECT AND GRIND CUT SURFACE PER HOAN RETROFIT DETAILS FOR DISTRICT 11-0.
2. VERTICALLY LOCATE 2\(\phi\) HOLE ON THE \(\phi\) OF THE LONGIT. STIFF.

DETAIL B1
SCALE: 3'=1'-0"

PLAN VIEW
SCALE: 3'=1'-0"

NOTES:
VERTICALLY LOCATE CORED HOLE ON THE \(\phi\) OF THE LONGIT. STIFF.

DETAIL B2 - G<1/4" ONE SIDE ONLY
SCALE: 3'=1'-0"

NOTES:
VERTICALLY LOCATE CORED HOLE ON THE \(\phi\) OF THE LONGIT. STIFF.

DETAIL B3 - G<1/4" BOTH SIDES
SCALE: 3'=1'-0"
ELEVATION - TIE GIRDER
SCALE: 3/16"=1'-0"

SECTION C-C
SCALE: 1/4"=1'-0"

NOTES:
1. CORE TO BE DRILLED FROM THE OUTSIDE.
2. CORE SHALL CAPTURE THE ENTIRE PROFILE OF WELDS BTW. LONGIT. AND TRANSV. STIFF. R.

DETAIL D
SCALE: 1-1/2"=1'-0"

DETAIL E
SCALE: 1-1/2"=1'-0"

LONGITUDINAL STIFFENER R1/2x6, TYP.
DIAPH. R 1/2"
12x20 ACCESS HOLE

TRANSVERSE STIFFENER R1/2, F.S.
LONGITUDINAL STIFFENER R1/2, F.S.
2-1/2Ø MIN. HOLE
1/8Ø REFERENCE HOLE

EDGE OF CORED HOLE
LONGITUDINAL STIFFENER R1/2x6
DRILLED HOLE (DRILL 4 TOTAL) DIAM. AS REQ'D CUT BETWEEN HOLES TO REMOVE CORE

OUTSIDE GIRDER WEB R1/2
SEALING DIAPHRAGM
TBD

TIED ARCH
CORE SAMPLE DETAILS
NOTE:
1. IF CORE DAMAGES THE GIRDER FLANGE, IT SHALL BE REPAIRED TO THE APPROVAL OF THE ENGINEER.

DETAIL F - INSIDE LOOKING OUT
SCALE: 1-1/2"=1'-0"
Appendix E

Evaluation of Bent Cap, Ramp-B
E1. Introduction
E1.1 Bridge Description – Bent Cap
A steel bent cap, present at many locations throughout the complex, was selected for instrumentation in order to quantify the stress ranges at some questionable welds. The bent in question, shown in Figure E.1, carries ramp B over Ohio River Blvd. onto the southbound portion of the main tied arch span of the Neville Island Bridge. The ramp is a 4-span continuous composite curved steel deck girder bridge. The girders are set on a steel bent cap, which is connected to the pier by means of threaded rods. The cap consisted of steel plates welded together. Strain gages were installed at various locations on the cap to measure the stress-range cycles.

Figure E.1 – Curved Girder Bridge and instrumented bent cap

E2. Instrumentation Plan and Data Acquisition
The following section describes the instrumentation plan used for the long term monitoring. Controlled load tests were not performed. A detailed instrumentation plan can be found in Appendix A.

E2.1 Strain Gages
The location of the strain gages was chosen such that the overall response of the pier cap could be established. Weldable uniaxial strain gages produced by Measurements Group Inc. were used. The gages were type LWK-06-W250B-350 with an active grid length of 0.25 inches. These gages are uniaxial temperature-compensated strain gages.

E2.2 Summary of Instrumentation Layout – Bent Cap
The following section summarizes the instrumentation plane. Detailed instrumentation plan is provided in Appendix A.
E2.2.1 Strain Gages on Top and Bottom Plates of Cap
Gages were installed close to the corner of the top and bottom faces of the steel flanges of the bent cap where it meets the pier. The bent beam is essentially a box beam. Figure E.2 shows a strain gage mounted to the underside of the steel bent cap.

Figure E.2 – Strain gage (CH_6) installed on the bottom face of the bent cap

E3. General Response
The response of all strain gages to normal traffic was very low. In general, stresses remained below 1 ksi at all channels. As an example, Figure E.3 shows the response of CH_3, CH_4, CH_5, and CH_6.
Figure E.3 – Typical stress time-history for strain gages mounted to the top (CH_3 and CH_4) and bottom (CH_5 and CH_6) of the bent cap.