FORENSIC INVESTIGATIONS OF THE INNERBELT BRIDGE (CUY-90-1524)

IN CLEVELAND, OHIO

by

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Submitted in partial fulfillment of the requirements

For the degree of Doctor of Philosophy

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__October 28, 2010_____

*We also certify that written approval has been obtained for any proprietary material contained therein.
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Forensic Investigations of The Innerbelt Bridge (Cuy-90-1524) in Cleveland, Ohio

Abstract

by

JOHN C. CLEARY

The Innerbelt Bridge (CUY-90-1524) carries Interstate Route 90 over the Cuyahoga River Valley in Cleveland, Ohio. The bridge was opened to traffic in 1959 and is a cantilevered steel deck truss with nine spans for a total length of 2,721 feet and a main span of 400 feet. Historically there have been several structural concerns with this particular bridge. Some of the concerns that have been noted are section loss due to corrosion, slope stability of the west bank, thermal strains due to inoperable expansion joints, and possible fatigue concerns due to out-of-plane vibrations in the web of a plate girder. The plate girder was added to widen the west end of the south side of the bridge.

The research presented here will discuss four investigations into the condition of the bridge. The first investigation consisted of instrumentation and live load testing of several members. The purpose of the investigations was to assess the condition of the members and to validate a finite element model of the bridge. An investigation into out-of-plane vibrations in the web of a plate girder was then conducted. This investigation’s aim was to determine the cause of and severity of the vibrations. The third investigation consisted of monitoring the structure during repairs and modifications. Due to slope
movement, the expansion joint located in Span 2 had “closed” and was inoperable. Several members were monitored during operations to “open” the expansion joint. The final investigation also involved structural monitoring. Several members were instrumented during the removal of the previously mentioned plate girder and the changes in the loads on the bridge were calculated. The results of this work were used to determine the condition and to aid in developing load ratings for the bridge. This work shows the importance of structural monitoring and investigation.
CHAPTER 1: INTRODUCTION

1.1 Forensic Investigations

Forensic engineering or forensic investigations are often thought of as the investigations of failures (Feld & Carper, 1997). However, forensic investigations include the investigation of structures to determine and assess their condition. Investigations can be performed on structures that are of concern to owners or structures that are being considered to be upgraded or used for purposes other than originally designed. In addition to aforementioned applications, inspections can be used on regular intervals to ensure the safety and monitor changes in the structure.

Much of our nation’s infrastructure is near the end or has exceeded its design life. In the case of bridges, which are typically designed for a 50 year life span, the average age is 43 years old (AASHTO, 2008). According to the U.S. Department of Transportation as of December 2008 there were 600,905 bridges across the nation. Of those bridges 12.1% or 72,868 were classified as structurally deficient and 14.8% or 89,024 were functionally obsolete (ASCE, 2009). In the report card for our nation’s infrastructure issued by the American Society of Civil Engineers (ASCE) our overall infrastructure system was given a “D” and our bridges received a “C” (ASCE, 2009). Many factors went into the grading system, but it is evident that our infrastructure system needs to be improved.

The cost to repair all of our nation’s deficient bridges is estimated at $140 billion (AASHTO, 2008). Since the funds to repair all of the deficient bridges is unavailable, investigations into individual bridges need to be conducted to evaluate their condition and determine what repairs are critical to ensure safety. Forensic investigations are also
needed to determine which bridges are critical and to prioritize and allocate the available money in the most efficient manner.

All of our nation’s bridges are required to be inspected at routine intervals not to exceed two years (FHWA, 2004). Many of these inspections, however, are performed by visual techniques only (Helmicki, Hunt, & Swanson, 2006). Although this may be sufficient for a majority of the bridges in this country many bridges require more in-depth investigations. Bridges that fall into this category include structures with fracture critical members. Fracture critical members are defined as components whose failure results in the collapse of the bridge or the inability of the bridge to perform its intended function (AASHTO, 2004). Approximately 11% of the bridges in the United States are considered fracture critical (Conner, Dexter, & Mahmoud, 2005). Fracture critical bridges may not have been properly designed for fatigue or section loss due to corrosion. Also, these designs may not have properly considered the increased traffic loads that are currently in service. Due to the potential danger of sudden collapse more frequent and thorough inspections are warranted for these structures.

Forensic investigations can utilize many techniques depending on the requirements for the individual structure. These may range from the previously mentioned visual inspections to instrumentation and monitoring, live load testing, and structural modeling. There is a wide range of nondestructive and destructive tests that can be performed. Some common nondestructive tests include, but are not limited to, acoustic emissions, dye penetrate, and magnetic particle testing, hardness testing, and strain gauge monitoring. Some destructive testing includes coring concrete samples and performing strength evaluation tests, as well as, obtaining steel samples and performing
tensile strength tests, Charpy impact tests, and others. Not all of the mentioned
techniques and tests need to be performed on every structure; however a qualified
investigator should be consulted to determine the appropriate tests to be performed.

1.2 The Innerbelt Bridge

The Innerbelt Bridge, CUY-90-1524, is located in Cleveland, Ohio and carries
Interstate 90 over the Cuyahoga River Valley. The bridge was opened to traffic in 1959
and carries approximately 134,000 vehicles a day. The design of the bridge is a
cantilevered steel deck truss consisting of nine spans with a total length of 2,721 feet (829
meters). The main span, Span 2, is located over the Cuyahoga River and is 400 ft (122
meters) long; Figure 1 shows a schematic diagram of the bridge, looking north and Figure
2 shows an expanded view of the river span. The bridge has twin trusses spaced at 90 ft
(27 meters) and an overall width of 116 ft (35.4 meters). There is a horizontal curve with
a central angle of approximately 21 degrees on a 1,400 ft (427 meter) section. The curve
begins in Span 4 and ends in Span 8. The bridge has a 340 foot (104 meter) long vertical
curve in Span 6. The bridge is considered fracture critical in that the failure of a single
member or connection could cause a catastrophic failure of the entire structure.

Figure 1: Schematic diagram of the bridge (ODOT, 1955).
In 1984 the bridge was widened to add an east-bound entrance ramp at the west end of the bridge, allowing traffic from West 14th street to enter the interstate. To facilitate the widening a 10 foot (3.05 meter) deep stiffened plate girder was added to the south side of the bridge, Figure 3. The girder has 5 continuous spans for a total length of 525 feet (160 meters) starting at the west end abutment. The widened portion of the roadway varies from 23 feet (7 meters) at the west end abutment to essentially zero at the end of the widening girder.
Historically this particular bridge has had many structural concerns. Richland Engineering Limited (REL), a Northeast Ohio based design firm, was retained in 1970 to analyze and monitor the structure. Since that time REL along with several other design firms have been working on the bridge. One of the concerns that the bridge has had is corrosion. Large amounts of deicing salts are used in the region to control ice on the roadways and the result is saltwater running over the bridge. This causes corrosion and section loss to both critical and noncritical members as shown in Figure 4.

The bridge has also had slope stability failures at the west end of the bridge. In the early 1970’s it was observed that the expansion joints in Span 2 were not operating properly and were “closed”. In 1973 the expansion joints were rebuilt to allow proper operation. During the early 1980’s it was suspected that the slope, where the west end abutment and Pier 1, are located was sliding (soil creep) to the east. This soil movement was causing the expansion joint in Span 2, the river span, to close, Figure 2. In 1995 drainage was added and slope grading was performed in an attempt to slow the soil creep. Then in 1999 a slope stabilization project was completed and Units 1 and 2 (see Figure 2), located in Spans 1 and 2, were moved several inches towards the west to open the expansion joints in Span 2.

Extensive structural monitoring, as well as, consulting and research projects have been conducted on this structure. Since 1991 several instruments have been in continuous use. This includes inclinometers and survey monuments to monitor soil movement, piezometers to monitor water levels, and tilt meters to monitor the rotations in Pier 1. Several of the studies conducted include: load ratings, fatigue analysis, live load testing, and monitoring during structural repairs.
1.3 Historic Research Projects

As mentioned above many research projects have been conducted on the Innerbelt Bridge. Several of these projects were related to the slope movement, expansion joint opening, and the slope stabilization project. Prior to the installation of the slope stabilization structure and the opening of the expansion joint several instruments were installed to monitor the movement of the slope and the rotation, or tilt, of the foundation piers. These instruments included inclinometers to monitor slope movements and tilt meters to measure rotations of the foundation piers.

1.3.1 Slope Stabilization Project

Prior to construction of the stabilization structure eight inclinometers were installed and monitored. Figure 5 shows the locations of all inclinometers used throughout the duration of the slope stabilization project. Inclinometers B-101 through B-105, B-107, B-108, and B-110 were installed and monitored between 1997 and 1999,
prior to the start of construction. During construction two of the inclinometers, B-103 and B-105 became unreadable due to large displacements that sheared the shaft casings. Three more inclinometers were added at that time; B-203, B-204, and B-303 (BBC&M Engineering, Inc., 2005).

Two failure planes exist in the vicinity of the Innerbelt Bridge. The first is located at approximately 40 feet below the surface and the other is at approximately 100 feet below the surface. Table 1 shows the average rate of movement for each of the locations instrumented for both the shallow and deep failure planes.

![Figure 5: Locations of inclinometers (BBC&M Engineering, Inc., 2005).](image-url)
<table>
<thead>
<tr>
<th>Location</th>
<th>Prior to Construction</th>
<th>During Construction</th>
<th>After Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Just outside of Pier 1 area</td>
<td>0.05</td>
<td>1.02</td>
<td>0.06</td>
</tr>
<tr>
<td>Well outside of Pier 1 area</td>
<td>0.03</td>
<td>0.03</td>
<td>0.16</td>
</tr>
</tbody>
</table>

**Shallow Failure Plane**

<table>
<thead>
<tr>
<th>Location</th>
<th>Prior to Construction</th>
<th>During Construction</th>
<th>After Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>South of west end pier</td>
<td>0.05</td>
<td>0.05</td>
<td>0.02</td>
</tr>
<tr>
<td>Just outside of Pier 1 area</td>
<td>0.08</td>
<td>0.78</td>
<td>0.10</td>
</tr>
<tr>
<td>Inside of Pier 1 area</td>
<td>N/A*</td>
<td>0.21/2.92/0.37**</td>
<td>0.06**</td>
</tr>
<tr>
<td>Well outside of Pier 1 area</td>
<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
</tr>
</tbody>
</table>

* No inclinometers were installed prior to construction
** Rates changed significantly during construction

Table 1: Average rate of movement from inclinometers (BBC&M Engineering, Inc., 2005).

It can be observed that the inclinometers located inside or just outside of the Pier 1 area had an increase in slip rate during construction. The deep failure plane then had a decrease in slip rate after construction was completed. The inclinometer well outside of Pier 1 had no change in slip rate during construction; however, the shallow failure plane had an increase after construction was completed.

It is interesting to note that what appears to have happened for the shallow slip plane is that the rate of slip in front of the piers was reduced, but the slip rate around the piers increased. The soil appears to be “arching” across the stabilization structure and flowing around it, Figure 6. Overall it appears that the slip rate of the deep failure plane was successfully reduced. However, the increased slip rate of the shallow failure plane around the structure may be of concern.
1.3.2 Truss Monitoring

Tiltmeters were installed on the north and south pier leg of Pier 1 in 1998. Data was collected from September 1998 to August 1999 on a daily basis for rotations in the north-south and east-west directions. At that time the expansion joint was opened to reduce thermal strains. After the completion of expansion joint operations, tilt meter data was collected on a quarterly basis. In 2005 strain gauges and thermistors were installed on the truss members framing into Pier 1. Figure 7 shows a timeline for the repairs and monitoring of the structure. Figure 8 shows the locations of the tiltmeters, strain gauges, and thermistors. A relationship could be developed between rotation of the pier leg, thrust from axial strains in the truss members, and temperature changes in the truss members. The results presented below are from: (Huckelbridge, 2005).
Figure 7: Timeline of repairs and studies.

Figure 8: Locations of tiltmeters, strain gauges, and thermistors (Huckelbridge, 2005).
Prior to the opening of the expansion joint, the north leg of the pier experienced a rotation of approximately 25 microradians per degree Fahrenheit. The south leg rotated approximately 35 microradians per degree Fahrenheit. This correlates to a horizontal thrust load of approximately 27 kip/°F in the north leg and 10 kip/°F in the south leg. The direction of the horizontal thrust and the resulting deflection of the bridge pier are shown in Figure 9.

![Figure 9: Deflection of foundation pier (Huckelbridge, 2005).](image)

During the opening of the expansion joint the lateral restraint at the top of the pier was removed and the North pier leg “straightened”, top of pier moved towards the east, by 2200 microradians or 0.91 inches (23.1 mm) at the top of the pier. The South pier leg
“straightened” by 3600 microradians or 1.78 inches (45.2 mm) at the top of the pier when the restraint was released. These rotations imply an unloading of 2420 kips (10764.7 kN) and 1040 kips (4626.2 kN) for the North and South trusses, respectively.

After the opening of the expansion joint a response of 6 microradians per degree Fahrenheit was observed in both pier legs. This correlates to a thrust load of 1.7 kip/°F in the south leg and 6.6 kip/°F in the north leg. However, by 2004 it was observed that large thermally-induced tilts were again occurring. This would indicate that the expansion joint was again partially closed. Since the installed inclinometers indicated a slower rate of soil creep the primary cause of the expansion joint closing was thought to be creep recovery in the piers and substructure.

Prior to the expansion joint opening, the piers had been restrained at the top and the bottoms continued to move. This introduced bending strains into the concrete piers, as shown in Figure 9. The concrete piers then experienced creep strains over a long period of time. Then after the lateral restraint was removed and the initial elastic recovery had taken place creep recovery occurred in the concrete. This creep recovery contributed to the closing of the expansion joint.

At the time of the study (Huckelbridge, 2005) it was determined that both pier legs were experiencing thermal thrust loadings of 8 kips per degree Fahrenheit. The south pier experiences more rotation than the north pier, 25 microradians per degree Fahrenheit versus 10 microradians per degree Fahrenheit. The study also determined that the south truss would experience an additional thrust of 29 to 58 kips per year due to the continued soil creep. The north truss is estimated to experience an additional thrust of 130 to 260 kips per year. The above increase in thrust assumes that the creep recovery in
the concrete was complete and that the soil would continue to creep at a rate of 0.05 to 0.10 inches per year. See (Huckelbridge, 2005) for all computations.

The final conclusion of the study was that the south and north piers could expect to see an additional of 1000 and 2700 kips of thrust, respectively, due to thermal stresses in the truss. The above estimates assume an annual temperature range of 100°F and the expansion joints to be inoperable. If the above relationships hold true and it is assumed that the pier legs cannot sustain an axial thrust greater than 5000 kips, then additional action may need to be taken within 7 to 10 years. It was also noted that an analysis of the truss should be performed to determine if the above thrust forces could cause structural failure.

1.4 Current Research

Several investigations were conducted and are reported here. The first was an investigation into the response of the structure to live load testing. Several members were instrumented with strain gauges and live load tests were performed using dump trucks loaded with salt. The instrumented members were located in two key areas of the bridge. The first was in Span 2 near the L300-L301 member, which is just east of the expansion joint, on the south truss. The second member instrumented was in Span 8, also in the south truss. The instrumented locations were chosen by individuals from the Ohio Department of Transportation (ODOT) based on results from a finite element model of the bridge developed by Michel Baker Jr. Inc., a private consulting firm with an office in Cleveland. This project was used to validate the finite element model and to evaluate the condition of the instrumented members.
The second investigation involved instrumentation of the widening girder used to facilitate the addition of the eastbound entrance ramp at West 14th street. During routine inspections large out-of-plane deformations were observed in the web of the plate girder. The member was instrumented with strain gauges and accelerometers, and then live load testing was performed. This testing was also accomplished with dump trucks loaded with salt. The objective of this investigation was to determine the cause of and severity of the out-of-plane vibrations. Several finite element models were also developed to predict the behavior of the plate girder.

The next investigation consisted of the monitoring of critical members during the opening of the expansion joint. As previously mentioned soil movements had closed the expansion joint located in Span 2. A section of the bridge was moved several inches towards the west to reopen the expansion joint. During relocation operations several members were instrumented and monitored. Truss members L27-L28 and L300-L301 in both the north and south trusses were instrumented with strain gauges and thermistors. These members are in the lower chord of the truss on the west and east side of the expansion joint. In addition to the instrumented members displacement transducers were located near the top and bottom line of the trusses. Tiltmeters and thermistors were also located on the north and south legs of Pier 1. The objective of this study was to monitor of members to ensure critical stresses, provided by the design engineer, were not reached. The displacements of the bridge were also monitored.

The final study discussed here involved the monitoring of the bridge during the removal of the widening girder. After the expansion joint opening was completed ODOT officials determined that it was necessary to remove the plate girder that was installed in
the early eighties to facilitate the addition of the W14th street entrance ramp. The strain
gauges that were installed to monitor the opening of the expansion joint were utilized to
monitor changes in strain due to the removal of the widening girder. The purpose of this
study was to determine the effect of removing the widening girder on the instrumented
members. The paper is organized such that each subsequent chapter discusses each
project followed by a section discussing the overall conclusions of the projects.
CHAPTER 2: LIVE LOAD TESTING

2.1 Introduction

In the fall of 2009 researchers from Case Western Reserve University in partnership with researchers from the University of Cincinnati performed live load testing on the Innerbelt Bridge to validate a three dimensional finite element model prepared by Michael Baker, Inc.’s Cleveland office and Richland Engineering Limited (REL). Two portions of the bridge were instrumented, one in Span 2 and the other in Span 8.

The results of the finite element analysis indicated that member L300-L301, Span 2, and member L717-L718, Span 8, both in the bottom chord of the south truss were overstressed under design loads. Figure 10 shows the approximate locations of the above mentioned members. The bridge was instrumented and live load testing was performed. The results were then compared with the computer model to verify and improve the accuracy of the output.

Figure 10: Approximate location of instrumented members.
2.2 Procedure and Test Methods

A total of fifty-two strain gauges were installed at the two locations. Thirty-two of the gauges were located near the L300-L301 member. Figure 11 thru Figure 14 display the locations of all of the strain gauges located near member L300-L301. Figure 15 and Figure 16 show the locations of all of the strain gauges near member L717-L718 as reported by: (Helmicki, Hunt, Swanson, & Huckelbridge, 2008). The gauges were installed by grinding the members to bare metal at the gauge location and bonding the gauges to the members with a fast setting gel based adhesive.

Figure 11: Instrumented locations at L300-L301.

Figure 12: Strain gauge locations at sections “A” and “C” in L300-L301.
The gauges were connected to an Optim Megadac data acquisition system. The data acquisition system at member L300-L301 was located on the catwalk adjacent to the member. At member L717-L718 the system was located on the sidewalk on top of the bridge. Both systems were controlled by a laptop computer and samples were collected at 200 samples per second or hertz (Hz). Each system also had a triggering mechanism to mark the passage of the trucks over each pier. Each truck had a two way radio to communicate the location of the truck to the data collection operator.
Live load testing was performed in October of 2008. Testing was accomplished with the use of Ohio Department of Transportation (ODOT) three axle dump trucks. Table 2 shows the wheel weights and Figure 17 shows the dimensions of the three trucks used during testing. There were a total of fifteen tests performed, eleven dynamic and four static.
<table>
<thead>
<tr>
<th>License Number</th>
<th>Front Axle</th>
<th>Middle Axle</th>
<th>Rear Axle</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Left kips</td>
<td>Right kips</td>
<td>Left kips</td>
<td>Right kips</td>
</tr>
<tr>
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<td>7.95</td>
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<td>7.00</td>
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<td>9.20</td>
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<td>7.75</td>
<td>7.35</td>
<td>7.65</td>
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</table>

Note: all trucks are International model 7400 SFA 6x4 with snow plow mounted and loaded with salt.

Table 2: Test truck wheel weights.

The configuration of the live load testing is shown in Table 3 and the lane designations are shown in Figure 18. Table 4, Figure 19, and Figure 20 show the configuration and locations of the trucks during the static load tests.

Figure 17: Dimensions of test truck.

Figure 18: Lane designations for live load testing (note: north is at top of figure).
### Table 3: Location and configuration of live load tests.

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Truck License Number</th>
<th>Position (Eastbound)</th>
<th>Speed</th>
<th>Notes</th>
</tr>
</thead>
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<td>Lane # 4</td>
<td>≈ 30 MPH</td>
<td></td>
</tr>
<tr>
<td>ML 2</td>
<td>T 12557</td>
<td>Lane # 3</td>
<td>≈ 30 MPH</td>
<td></td>
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<tr>
<td>ML 3</td>
<td>T 12557</td>
<td>Lane # 3</td>
<td>≈ 30 MPH</td>
<td></td>
</tr>
<tr>
<td>ML 4</td>
<td>T 12557</td>
<td>Lane # 2</td>
<td>≈ 30 MPH</td>
<td></td>
</tr>
<tr>
<td>ML 5</td>
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<tr>
<td>ML 7</td>
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</tr>
<tr>
<td>ML 8</td>
<td>T 12557</td>
<td>Lane # 1</td>
<td>≈ 30 MPH</td>
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</tr>
<tr>
<td>ML 9</td>
<td>T 12557</td>
<td>Lane # 1</td>
<td>≈ 45-50 MPH</td>
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</tr>
<tr>
<td>ML 10</td>
<td>T 12557 (Lead)</td>
<td>Lane # 4</td>
<td>≈ 30 MPH</td>
<td>Missed trigger for pier #4 test was repeated as ML 11</td>
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<tr>
<td></td>
<td>T 12549 (Middle)</td>
<td>Lane # 4</td>
<td>≈ 30 MPH</td>
<td></td>
</tr>
<tr>
<td></td>
<td>T 12844 (Trailing)</td>
<td>Lane # 4</td>
<td>≈ 30 MPH</td>
<td></td>
</tr>
<tr>
<td>ML 11</td>
<td>T 12557 (Lead)</td>
<td>Lane # 4</td>
<td>≈ 30 MPH</td>
<td></td>
</tr>
<tr>
<td></td>
<td>T 12549 (Middle)</td>
<td>Lane # 4</td>
<td>≈ 30 MPH</td>
<td></td>
</tr>
<tr>
<td></td>
<td>T 12844 (Trailing)</td>
<td>Lane # 4</td>
<td>≈ 30 MPH</td>
<td></td>
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### Table 4: Location and configuration of static load test.

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<th>Test Number</th>
<th>Truck License Number</th>
<th>Position</th>
<th>Distance from Rear Axle to Expansion Joint Centerline</th>
<th>Distance from Curb to Centerline Wheel</th>
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<td>80 in</td>
<td>30 ft 10 in</td>
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<td></td>
<td>T 12844</td>
<td>Lane # 3</td>
<td>65 in</td>
<td>17 ft 6 in</td>
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<td></td>
<td>T 12549</td>
<td>Lane # 4</td>
<td>75 in</td>
<td>4 ft 8 in</td>
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<td>SL 2</td>
<td>T 12557</td>
<td>Lane # 2</td>
<td>73 in</td>
<td>16 ft 6 in (Median face)</td>
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<td>T 12844</td>
<td>Lane # 3</td>
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<td>28 ft 6 in (Median face)</td>
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<tr>
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<td>T 12549</td>
<td>Lane # 4</td>
<td>72 in</td>
<td>40 ft 8 in (Median face)</td>
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<tr>
<td>SL 3</td>
<td>T 12557</td>
<td>Lane # 3</td>
<td>71 in</td>
<td>17 ft 2 in</td>
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<tr>
<td></td>
<td>T 12844</td>
<td>Lane # 3</td>
<td>68 in</td>
<td>30 ft 7 in</td>
</tr>
<tr>
<td></td>
<td>T 12549</td>
<td>Lane # 4</td>
<td>75 in</td>
<td>4 ft 2 in</td>
</tr>
<tr>
<td>SL 4</td>
<td>T 12549</td>
<td>Lane # 4</td>
<td>71 in</td>
<td>4 ft 2 in</td>
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<td></td>
<td>T 12844</td>
<td>Lane # 3</td>
<td>75 in</td>
<td>16 ft 9 in</td>
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<tr>
<td></td>
<td>T 12557</td>
<td>Lane # 2</td>
<td>70 in</td>
<td>30 ft 7 in</td>
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2.3 Dynamic Load Test Results

As mentioned a total of eleven dynamic (moving) load tests were conducted. Of those tests three were repeated due to observed drift and difficulties with the triggering mechanism. Therefore only the results of eight of the dynamic load tests will be discussed here. Table 5, Table 6, and Table 7 contain a summary of the maximum strain from all of the dynamic tests. For the strain gauges on member L300-L301, the largest strains were due to moving load test 7 and 11. Moving test 7 consisted of the three trucks driving side-by-side near the curb and moving test 11 consisted of a three truck train in the curb lane. The maximum strain in any of the gauges in Span 2 throughout testing was 59.2 microstrain (με). This strain occurred in gauge A9, . The largest strains in each of the members connected to L301 were 41.1 με in gauge E5940 on the vertical, 42.3 με in gauge F5942 on the diagonal, and 34.3 με in gauge D2 on the lower chord member. The
strain gauges on member L717-L718 had the largest strains due to moving load 7 and 8.

Both tests consisted of the three trucks driving side-by-side with ML 7 closest to the curb and ML 8 closest to the median. The largest strain in L717-L718 was recorded on gauge A10 and had a magnitude of 48.6 με.

<table>
<thead>
<tr>
<th>Strain Gauge</th>
<th>ML 1</th>
<th>ML 2</th>
<th>ML 3</th>
<th>ML 4</th>
<th>ML 5</th>
<th>ML 6</th>
<th>ML 7</th>
<th>ML 8</th>
<th>ML 9</th>
<th>ML 11</th>
<th>Max (test)</th>
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Table 5: Maximum strain for strain gauges on L300-L301.

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<th>Strain Gauge</th>
<th>ML 1</th>
<th>ML 2</th>
<th>ML 3</th>
<th>ML 4</th>
<th>ML 5</th>
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<th>Max</th>
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</table>

Table 6: Maximum strain for strain gauges connecting to L301.
<table>
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<th>Strain Gauge</th>
<th>ML 1</th>
<th>ML 3</th>
<th>ML 4</th>
<th>ML 6</th>
<th>ML 7</th>
<th>ML 8</th>
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**Table 7: Maximum strain for strain gauges on L717-L718.**

Figure 21 shows a representative plot of the strain in the L300-L301 members, the line across the top of the plot indicates when the test truck passed over each pier. It can be observed that as the truck approached Pier 1 the member went into tension (came out of compression) then went into compression as the truck passed Pier 1. A frequency analysis was performed by applying a Fast Fourier Transform to the strain gauges from each section, Figure 22 and Figure 23. The frequency analysis of section A shows two dominant frequencies, approximately 2 and 3.5 hertz (Hz). Section C has only one dominant frequency of 3.5 Hz. The bridge is known to have a fundamental frequency of 3.5 Hz, from prior testing. The 2 Hz frequency appears in the analysis of sections D, E, and F; although not as strong. It is unknown where this frequency signature is originating from.
Figure 21: Strain in member L300-L301 during dynamic load testing.

Figure 22: Frequency analysis of L300-L301 section A.
A representative plot of the strain in member L717-L718 can be seen in Figure 24, the vertical lines near the top of the plot indicate when the truck passed over each pier. Figure 25 and Figure 26 show the results of a frequency analysis on sections A and C. There is no dominant frequency that can be determined from this analysis and several “spikes” are seen between 2.5 and 3.5 Hz. Although in section A there is a stronger frequency at 3.2 Hz. The fundamental frequency in this region of the bridge is expected to be different due to several factors. One reason is that the length of Span 2 is different from Span 8. Another reason is that the expansion joint in Span 8 is fully operational, unlike the expansion joint in Span 2.
Figure 24: Strain in member L717-L718 during dynamic load testing.

Figure 25: Frequency analysis of L717-L718 section A.
Figure 26: Frequency analysis of L717-L718 section C.

Figure 27 and Figure 28 show the dynamic response of members L300-L301 and L717-L718. For member L300-L301 the static response is approximately 17 microstrain and the peak response is 19 microstrain. This correlates to a dynamic response of 2 microstrain. The dynamic amplification factor of member L300-L301 is approximately 1.12. The static response in member L717-L718 is approximately 19 microstrain and the peak response is 24 microstrain. The dynamic response of member L717-L718 is 5 microstrain and the dynamic amplification factor is 1.26.
Figure 27: Dynamic response of member L300-L301.

Figure 28: Dynamic response of member L717-L718.
2.4 Static Load Test Results

Four static load tests were performed, as discussed previously. Since two of the load tests were near member L300-L301 and two were near L717-L718 only one data collector was used at a time. For static test 1 and 3 the data collector near L717-L718 was used and for static test 2 and 4 the data collector near L300-L301 was used. Table 8 displays the results of the average strain gauge readings from the static tests. The maximum strain in member L300-L301 was in gauge A9 and had a magnitude of 52.9 $\mu$ε. For the members connected to L301 the maximum strains were 43.6 $\mu$ε in gauge E5940 of the vertical, 39.6 $\mu$ε in gauge F5942 of the diagonal, and gauge D2 of the lower chord member had a strain of 32.1 $\mu$ε. The maximum strain in the L717-L718 member was 38.3 $\mu$ε in gauge A1. In comparing the static and dynamic strains, the maximum strains in members near L301 all came from the same strain gauge in each member. For the L717-L718 member the maximum strain from the dynamic loading was A10 and for the static loading was A1. For both of these loading cases the difference in strain between the two gauges is small.
### Table 8: Average strains during static load tests.

<table>
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<th>SL 2</th>
<th>SL 4</th>
<th>Strain Gauge</th>
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<th>SL 4</th>
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Note: Strains in **BOLD** are in tension, all others are in compression.

### 2.5 Plane Section Behavior

It can be observed from the static strain response that the assumption of plane section behavior does not appear to be occurring, Figure 29. An example of this observation can be shown by performing a least squares fit of the observed strain data to a plane section assumption. The bending strains/curvature observed and predicted is due to restraint at the end of the members. Although truss members are typically considered to be two force members, restraints can introduce bending strains. The restraint comes from the difference in an ideal connection, which would consist of a friction less pin, and the actual connections. The actual connections consist of gusset plates that the members
are connected to over a given distance. This connection introduces some amount of restraint that can induce bending.

If section A in member L300-L301 for static test two is used for the least squares fit then: from beam bending theory:

$$\begin{bmatrix} y_i \\ x_i \end{bmatrix} \begin{bmatrix} \Psi_{xx} \\ \Psi_{yy} \end{bmatrix} = \begin{bmatrix} \varepsilon_i - \varepsilon_{avg} \end{bmatrix}$$

Equation 1: Least squares fit from beam bending theory.

where: $y_i$ and $x_i$ represent the distances from the neutral axis to the point of interest and the x-direction is for strong axis bending

the least squares solution for curvatures is:

$$\begin{bmatrix} (y_i) \theta \\ (x_i) \theta \end{bmatrix} \begin{bmatrix} \Psi_{xx} \\ \Psi_{yy} \end{bmatrix} = \begin{bmatrix} (y_i) \theta \varepsilon_i - \varepsilon_{avg} \end{bmatrix}$$

Equation 2: Solution to least squares fit for curvature.

from Figure 12 and Table 8:

$$\begin{bmatrix} y_i \\ x_i \end{bmatrix} = \begin{bmatrix} -15.375 & 5 \\ -15.375 & -5 \\ -10.5 & 11 \\ -10.5 & -11 \\ 7.5 & 11.56 \\ 6 & -11.56 \\ 10.5 & 11 \\ 10.5 & -11 \\ -6 & 11.56 \\ -6 & -11.56 \end{bmatrix} \begin{bmatrix} \varepsilon_i - \varepsilon_{avg} \end{bmatrix} = \begin{bmatrix} -21.63 \\ -18.67 \\ 12.40 \\ -4.97 \\ 0.68 \\ 12.71 \\ 26.20 \\ 23.57 \\ -23.16 \\ -7.12 \end{bmatrix}$$

the results of the preceding calculations are:

$$\varepsilon_{avg} = -29.75 \mu \varepsilon$$
$$\Psi_{xx} = -1.233 \mu \varepsilon/in$$
$$\Psi_{yy} = 0.1316 \mu \varepsilon/in$$

Figure 29 and Figure 30 shows a comparison of the observed strains and the strains from the least squares plane section analysis for sections A and C in member L300-L301.
Table 9 contains the results of least squares plane section fit and the observed strain readings from each of the sections. From Figure 29 it can be observed that the measured strains do not appear to match the predicted strains from the plane section behavior theory.

Figure 29: Actual versus predicted strains for plane section behavior, section A.
Figure 30: Actual versus predicted strains for plane section behavior, section C.

Table 9: Actual versus predicted strains for plane sections behavior.

<table>
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<tr>
<th>Gauge</th>
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<tr>
<td>Ψyy</td>
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<td>Ψyy = 0.1702</td>
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<td>ε̃i - ε̃avg</td>
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The instrumented sections are at least one member depth away from the end connections, therefore stress concentrations should not be of concern. However, the members are built-up sections of plates and angles. The gauges with the highest strain readings from Figure 29 are the gauges that are located on the angles connecting the plates. Local stress concentrations from the rivets and the connection pieces could be causing what appear to be artificially high strains. Another possible cause of the distortion is local plate bending strains that are reducing the maximum compressive strains and influencing the average strains.

Figure 31 and Figure 32 display the least squares fit curvature for members L300-L301 and L717-L718. The data suggests that there is less bending strains in member L717-L718 than in L300-L301. This is most likely due to the configuration of the bridge. One interesting observation from the plots is the differences in average axial strain at the different locations of the same member. This may be due to the previously mentioned stress/strain concentrations and/or plate bending strains.

**Figure 31: Least squares fit curvature for L300-L301.**
2.6 Comparison of Measured and Predicted Response from FEM

The predicted average axial strains and least squares fit curvature from the finite element analysis performed by Baker were larger than the measured strains from the live load test. Table 10 shows the measured and predicted strains for both members. The ratio of the measured to predicted axial strain in member L300-L301 was at least 0.562. In member L717-L718 the ratio was at least 0.311. Literature distributed by Baker indicated that the predicted strains from the finite element model did take into account section loss due to corrosion. The predicted strains and curvature were consistently lower for the average strains, as well as, the strains for the individual gauges.
<table>
<thead>
<tr>
<th>Member</th>
<th>Section</th>
<th>Average Strain (με)</th>
<th>Least Squares Fit Strong Axis Curvature $\psi_{xx}$ (με/in)</th>
</tr>
</thead>
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</tr>
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</tr>
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</table>

Table 10: Measured versus predicted response for static load tests.

2.7 Conclusions

The purpose of this study was to verify and improve a finite element model of the structure. Static and dynamic tests were performed and results indicated that measured response was consistently lower than predicted response. For member L717-L718 the measured response for the static load test was approximately 39% of the predicted response for axial strains. The static response for axial strains in member L300-L301 was approximately 60% of the predicted response. The predicted curvature in the strong axis was also consistently higher than measured response. The measured response for member L717-L718 was 22.8% of the predicted curvature and for member L300-L301. For section A the measured response was 46% of the predicted response and the curvature had a different sense, positive instead of negative curvature for section C.

There are several factors that could affect the accuracy of a finite element model. One assumption that may have affected the results is the magnitude of the restraint at the connections. The smaller observed versus predicted curvature would indicate that the truss joints had less restraint than the finite element model assumed. The amount of section loss due to corrosion may have also been overestimated, to ensure conservative results. There are many other boundary conditions that could have affected the results. It
is important for design engineers to recognize the limitations of any model of a structure and realize that the model can output significantly different results depending on assumed boundary conditions. It is also important to understand that the model is only an approximation of the actual structure.

It was also observed that the plane section behavior did not appear to be valid. When closely analyzed the discrepancies in the measured strains can be explained by local stress/strain concentrations and plate bending strains. Since the members are made of plate sections connected by angles, local stresses at plate connections may be different than predicted response from plane section behavior. Also, in addition to the stress concentrations additional stresses may be occurring from local bending strains in the plate sections.

According to literature distributed by Richland Engineering Limited (REL) the instrumented members were overstressed from legal operating loads. Member L717-L718 needed the load to be reduced to 84% of the legal levels and member L300-L301 required the loads to be reduced to 62% to achieve acceptable stress levels. If the above findings can be linearly interpolated then the reduction in measured versus predicted strains for members L717-L718 and L300-L301 may indicate that the members would not be overstressed due to legal operating loads.
3.1 Introduction

In 1984 the Innerbelt Bridge was modified to accommodate an entrance ramp for eastbound traffic at West 14th street. The west end of the bridge needed to be widened to accommodate an acceleration lane. A plate girder was added to the south side of the bridge to facilitate the widening. The widened portion of the bridge varies from 23 feet (7.01 meters) at the west end abutment to essentially zero at the end of the girder.

The girder consists of 5 continuous spans with a total length of 525 feet (160 meters). The girder is supported at 6 locations, all of which, with the exception of the west end abutment, could be considered “elastic” in nature, Figure 33. The west end is supported with a roller joint allowing movement along the axis of the bridge. The girder is supported at floor beams 5, 14, 18, and 22 by a bracket system. The bracket supports the girder with a strut that goes from the girder to the bottom chord of the south truss and floor beam extensions, spanning horizontally from the top chord of the south truss to the widening girder, Figure 34. There is a bracket at floor beam 10 with a strut that goes from the bottom of the girder to Pier 1 and a floor beam extension.

![Diagram of Elastic supports of widening girder.](image)

1 Note: much of the work presented in this chapter had been presented and published elsewhere; see: (Cleary & Huckelbridge, 2010)
The floor beams are spaced at 25 feet (7.6 meters) and numbered sequentially starting at the west end abutment. The girder itself is a 10 foot (3 meter) deep stiffened plate girder, with a slender 0.375 inch (0.953 cm) web and variable flanges. Transverse stiffeners are located approximately every 6 feet (1.8 meters). A longitudinal stiffener is located near the Pier 1 bracket approximately 2 feet from the bottom flange.

During regular bridge inspections out-of-plane vibrations in the girder web, particularly adjacent to the Pier 1 bracket, were observed. These observations initiated an investigation to determine the cause, nature, and severity of the vibrations, and any potential corrective measures, should they be warranted. The investigation focused on the girder web panels immediately to the east of Pier 1, where vibrations had been noted, and access to the girder was facilitated by a permanent catwalk.
3.2 Procedure and Test Methods

Prior to live load testing, a finite element model of a portion of the widening girder was developed. The widening girder was modeled using SAP 2000 version 12. Dead load shears were provided by Michael Baker Jr, Inc.’s Cleveland office from a MIDAS 3-D model of the entire bridge structure, Figure 35. The section of the girder modeled was approximately 28 feet (8.5 meters) long and located adjacent to the Pier 1 support bracket. Transverse stiffeners were placed at 6 foot (1.8 meter) intervals in the web panel and a longitudinal stiffener was located 2 feet (0.6 meters) above the bottom flange. Figure 36 shows a screen shot of the finite element model, along with strain gauge placements for the monitoring program as reported in (Huckelbridge, 2009).

Figure 35: Shear stresses along the length of the widening girder (provided by Michael Baker, Inc.)
For live load testing both strain gauges and accelerometers were used. The main panel had a total of eight strain gauges installed. A strain gage rosette consisting of three individual gauges was located in the center of the panel, one transverse and one longitudinal gauge was located at the top of the panel, two longitudinal gauges were located at the edges of the panel, and one gauge was located at the longitudinal stiffener. In addition to the main panel, the support strut and floor beam extension were instrumented with three gauges each. The lateral braces were also instrumented with two strain gauges. All of the strain gauges used were HITEC model HBWF-35-125-6-xxGP-NT welded full bridge strain gauges.

In addition to the strain gauges, two removable magnetic base accelerometers (PCB Piezotronics Model 353B33) were used. The accelerometers were moved to different locations throughout the testing. Strain gauge and accelerometer data was collected at a sampling rate of 100 samples per second on a Campbell Scientific CR5000 data logger. Figure 37 shows a picture of the data collector.
Live load testing was performed in February of 2009. Testing was accomplished using two Ohio Department of Transportation (ODOT) three axle dump trucks loaded with salt. The total weight of the trucks were 48.5 kips (214.8 kN) and 43.55 kips (193.5 kN), Table 11 shows individual wheel weights. The trucks were driven on the entrance ramp and in the traveling lanes of the bridge in several configurations throughout testing.

<table>
<thead>
<tr>
<th>Truck Number</th>
<th>Front Axle Left</th>
<th>Front Axle Right</th>
<th>Middle Axle Left</th>
<th>Middle Axle Right</th>
<th>Rear Axle Left</th>
<th>Rear Axle Right</th>
<th>Total kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7.85</td>
<td>6.95</td>
<td>8.05</td>
<td>8.70</td>
<td>8.45</td>
<td>8.30</td>
<td>48.30</td>
</tr>
<tr>
<td>2</td>
<td>6.75</td>
<td>6.65</td>
<td>7.80</td>
<td>7.25</td>
<td>7.90</td>
<td>7.20</td>
<td>43.55</td>
</tr>
</tbody>
</table>

Table 11: Test truck wheel weights
3.3 **Initial Finite Element Modeling**

The dead load reactions, provided by Michael Baker, Inc., consisted of a shear load of approximately 288 kips (1281 kN) and a moment of 13,400 kip-ft (18,168 kN-m). From Figure 38 it can be seen that the in-plane shear stresses are between 6 and 7 ksi (41.4 and 48.3 MPa) (Huckelbridge, 2009).

![Figure 38: In-Plane shear stress contours.](image)

The elastic buckling stress can be calculated from the following equation (Salmon & Johnson, 1990):

\[
F_{cr} = k \frac{\pi^2 E}{12(1 - \mu^2)(\frac{b}{t})^2}
\]

**Equation 3: Critical buckling stress for plate element.**

where: \(F_{cr}\) = critical elastic buckling stress

- \(k\) = plate buckling coefficient
- \(E\) = modulus of elasticity of steel; 29,000 ksi (200,000 MPa)
- \(\mu\) = Poisson’s ratio for steel; 0.3
- \(b\) = distance between stiffeners; 72 in (183 cm)
- \(t\) = plate thickness; 0.375 in (0.953 cm)
The plate buckling coefficient with assumed knife edge supports can be calculated from the following (Salmon & Johnson, 1990):

\[
k = 5.34 + 4.0 \left( \frac{\text{short dimension}}{\text{long dimension}} \right)^2
\]

**Equation 4: Plate buckling coefficient.**

For the instrumented web panel the short dimension, between transverse stiffeners, is equal to approximately 6 ft (1.8 m) and the long dimension, between the tension flange and the longitudinal stiffener, is approximately 8 ft (2.4 m). These dimensions result in an estimated plate buckling coefficient of 7.59. Using this value of the plate buckling coefficient, the critical elastic buckling stress is approximately 5.5 ksi (37.9 MPa). These results were verified by performing a buckling analysis utilizing the finite element model. The results from both buckling analyses, Figure 39, indicate that the web panel would buckle at approximately 84% of the estimated dead load. A finite element model developed by Baker also concluded that the web panel would buckle under the dead load (Michael Baker Jr. Inc., 2009). Therefore, the web of the plate girder was assumed to be buckled under the dead load.

A modal analysis was also performed on the widening girder using the finite element model. The results, shown in Figure 40, from the analysis for an unbuckled web indicate that the two lowest out-of-plane frequencies would be approximately 15 hertz (Hz) and 26 Hz (Huckelbridge, 2009).
Predicting the natural frequencies for the in service widening girder is more difficult, since the web is assumed to be buckled under the dead load. A buckled web, however, would reasonably be expected to exhibit lower natural frequencies, due to the expected reduction in out-of-plane stiffness after buckling has occurred.
3.4 Field Testing Results

Field testing of the widening girder was performed in February of 2009. A total of fifteen test runs were completed, five of which had unusable data, due to several factors. Some of the test runs had a large amount of “noise” due to the passage of large trucks on the bridge, since the bridge was open to vehicular traffic throughout testing. Another problem with some of the data was the timing of the passage of the trucks and the data logger status, the data logger was downloading at the time of the truck passage. The ten tests with usable data, as well as, accelerometer and truck locations are shown in Table 12.

<table>
<thead>
<tr>
<th>Test</th>
<th>Filename</th>
<th>Acc A</th>
<th>Acc B</th>
<th>Truck</th>
<th>Position</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Table001</td>
<td>Center MP</td>
<td>Center Left Panel</td>
<td>1</td>
<td>Entrance Ramp: Curb (slow)</td>
</tr>
<tr>
<td>3</td>
<td>Table003</td>
<td>Center MP</td>
<td>Center Left Panel</td>
<td>1</td>
<td>Entrance Ramp: Curb (Fast)</td>
</tr>
<tr>
<td>4</td>
<td>Table004</td>
<td>Center MP</td>
<td>Center Left Panel</td>
<td>2</td>
<td>Entrance Ramp: Curb (Fast)</td>
</tr>
<tr>
<td>5</td>
<td>Table005</td>
<td>Center MP</td>
<td>Top Flange Main Panel</td>
<td>1</td>
<td>Entrance Ramp: Curb (Fast)</td>
</tr>
<tr>
<td>7</td>
<td>Table007</td>
<td>Center MP</td>
<td>Center Left Panel</td>
<td>Both</td>
<td>Traffic Lane Side by Side</td>
</tr>
<tr>
<td>8</td>
<td>Table008</td>
<td>Center MP</td>
<td>Center Left Panel</td>
<td>Both</td>
<td>Traffic Lane: Right Lane Train</td>
</tr>
<tr>
<td>9</td>
<td>Table009</td>
<td>Center MP</td>
<td>Center Left Panel</td>
<td>Both</td>
<td>Traffic Lane: Right Lane Train</td>
</tr>
<tr>
<td>11</td>
<td>Table011</td>
<td>FB Stiffener</td>
<td>Pin on Floor Beam</td>
<td>Both</td>
<td>East Bound High Speed Lane</td>
</tr>
<tr>
<td>12</td>
<td>Table012</td>
<td>FB Stiffener</td>
<td>Pin on Floor Beam</td>
<td>Both</td>
<td>West Bound High Speed Lane</td>
</tr>
<tr>
<td>14</td>
<td>Table014</td>
<td>Center MP</td>
<td>Center Left Panel</td>
<td>Both</td>
<td>Entrance Ramp: Curb (Fast)</td>
</tr>
</tbody>
</table>

Table 12: Test runs and trucks used.

3.4.1 Pseudo-Static Live Load Response

The first test run, labeled Table001, was the passage of a slow moving truck on the entrance ramp and across the bridge as close to the curb as possible. The truck used for this test was truck 1 and had total weight of 48.3 kips (214.8 kN).
Figure 41 and Figure 42 show the strains in the support strut and floor beam extension during this test run. It can be seen that the strut primarily goes into compression, but a small amount of bending does occur. The maximum amount of strain in any one gauge is 10 microstrain. The floor beam, however, has a larger amount of bending strain with a maximum strain of approximately 30 microstrain. Figure 43 displays the strains in the main panel during this test. The maximum amount of strain in the panel was in the strain gauge located adjacent to the longitudinal stiffener and was approximately 50 microstrain.

![Figure 41: Pseudo-static response of strut at Pier 1.](image)
Figure 42: Pseudo-static response of floor beam extension.

Figure 43: Pseudo-static strains in main panel.
It was observed during this portion of the testing that pseudo-static out-of-plane response of the web was occurring. This response correlated well with the applied pseudo-static live load shear in the web panel. Figure 44 shows the influence line for shear in the instrumented panel and the strain in the center transverse strain gauge, which should only register out-of-plane bending stresses. The results indicate that the most likely source of the observed out-of-plane response in the web of the widening girder is due to the applied live load shear. Since static live load shear produced out-of-plane response, due to the resulting increase in amplitude of the already buckled web panel, then dynamic shear loads would logically generate a dynamic out-of-plane response.

**Figure 44:** Pseudo-static out-of-plane response of the web due to live loads.
3.4.2 Dynamic Live Load Response

The dynamic live loads in the widening girder come from two distinct sources. The first is impact loads imparted on the girder due to the passage of vehicles on the W 14th street entrance ramp and the second comes from “support motion” or “seismic” excitations due to vehicles in the traveling lanes of the bridge. These vehicles cause displacements in the south truss, that then act as a “seismic” load on the supporting elements of the widening girder. Figure 45 and Figure 46 show the support structure and web panel reactions due to truck loads on the entrance ramp, see Figure 36 for strain gauge locations.

Figure 45: Support structure response due to dynamic live loads.
It was observed that the support structure has approximately the same amount of strain in both the pseudo-static and dynamic test. The web girder, however, has significantly less strain in the dynamic test then in the static test. The maximum strain, at the location of the longitudinal stiffener, is approximately half that of the static test. This may have been due to the location of the truck relative to the curb.

Figure 47 and Figure 48 show the strains in the top of the web panel (bending strains in the girder) and at the center of the main panel in the transverse direction (out-of-plane bending stresses). Both gauges show a definite 3 hertz (Hz) frequency signature and the gauge at the center shows higher frequency signatures as well. Figure 48 also shows the dynamic and static portions of the strains. The static and dynamic responses are approximately 10 microstrain and 12 microstrain, respectively. This correlates to a dynamic amplification factor of 1.2.
Figure 47: Bending strains in girder due to dynamic live loads.

Figure 48: Out-of-plane bending strains in web panel due to dynamic live loads.
Figure 49 and Figure 50 show a Fast Fourier Transform (FFT) of the main panel transverse gauge and the average of the support strut gauges. The web panel gauge shows frequency spikes at four locations; 3 Hz, 3.5 Hz, 11.5 Hz, and 22.7 Hz. The 3 Hz frequency is the assumed natural frequency of the widening girder since it is present in the support strut. The 3.5 Hz frequency is the natural frequency of the main bridge truss as confirmed in previous live load testing of this structure. The two higher frequency signatures in the web panel are assumed to be the first and second frequencies associated with the out-of-plane deformations in the web panel.
The observed out-of-plane natural frequencies, 11.5 Hz and 22.7 Hz, were less than the predicted “unbuckled elastic” frequencies of 15 Hz and 26 Hz from the finite element model. This result was reasonably expected, however, since there was a “softening” of the structural system when the web panel buckled under the dead load. There are no readily available models to determine the expected “buckled” natural frequencies; however, the observed trend is consistent with the expected outcome.

Table 13 shows the results of the frequency analysis of the center longitudinal strain gauge for all of the dynamic tests. The amount of variability in the test data is apparent. The average frequency for the first mode was 14.2 Hz and the second mode had an average frequency of 20.4 Hz. However, when the data was analyzed it was evident that Table014 had the strongest signal. For this reason the first and second modes are assumed to be the 11.5 Hz and 22.7 Hz reported previously.
Table 13: Out-of-plane frequencies in the web of the plate girder.

<table>
<thead>
<tr>
<th>Table</th>
<th>First Mode (Hz)</th>
<th>Second Mode (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>003</td>
<td>17.4</td>
<td>18.8</td>
</tr>
<tr>
<td>004</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>005</td>
<td>16.3</td>
<td>18.6</td>
</tr>
<tr>
<td>007</td>
<td>15.2</td>
<td>19.5</td>
</tr>
<tr>
<td>008</td>
<td>14.5</td>
<td>19.5</td>
</tr>
<tr>
<td>009</td>
<td>13.9</td>
<td>20.2</td>
</tr>
<tr>
<td>011</td>
<td>12.4</td>
<td>21.5</td>
</tr>
<tr>
<td>012</td>
<td>12.1</td>
<td>22.1</td>
</tr>
<tr>
<td>014</td>
<td>11.5</td>
<td>22.7</td>
</tr>
</tbody>
</table>

Figure 51 shows the results from the accelerometers during test run 14, with both trucks on the entrance ramp. For this test one accelerometer was in the center of the instrumented panel and the other one was in the center of the adjacent panel. It can be observed that the recorded accelerations in the web panel adjacent to the instrumented panel was greater than the acceleration in the instrumented panel.
The data from the accelerometers was used to approximate the amount of out-of-plane displacements in the web of the plate girder. First the acceleration data was integrated using the trapezoidal rule to determine the velocities. The velocity data was then corrected with the assumption of zero mean. Figure 52 shows the corrected velocity data.

![Figure 52: Corrected velocities in web panel from dynamic live loads.](image)

The velocity data was then integrated, again with the trapezoidal rule, and that data was again corrected using the zero mean assumption. Figure 53 shows the corrected displacement data.

The accuracy of the displacement data cannot be relied upon due to the simplified integrations and correction factors, as well as, the location of the instruments in relation to the buckled shape. However, several observations can be made. The first is that the magnitude of the displacements in the web panel to the east of the instrumented panel is approximately twice as large as in the instrumented panel. The second observation is that
the overall magnitude of the displacements is quite small; less than 3 mills (0.003 in). This is consistent with the data from the strain gauges, which indicated little out-of-plane deformations.

![Corrected displacements in web panel from dynamic live loads.](image)

The study by Baker did determine that the maximum out-of-plane deformations for an unbuckled web should be on the order of 0.1 inches (Michael Baker Jr. Inc., 2009). These results do not agree with the experimental results, however, the study by Baker did assume larger live loads then used during testing and an unbuckled web. The difference in loads and buckled state of the web may explain the discrepancy in testing and analytical results.

The maximum strain in each of the instrumented sections for all of the included test runs is shown in Table 14. It can be observed that the strains in the web panel, the support strut, and in the floor beam were highest during Test Run 001, which was the
pseudo static test. The lateral braces, however had larger stresses during test runs 007, 008, and 014. This may have been due to other traffic on the bridge since the strains are close together. It can be observed that all of the strains were small throughout testing. In particular the strains in the support strut were equal to or less than 10 microstrain. The stresses/strains in the strut were most likely designed to be low due to the long unbraced length, 70.5 ft (21.5 meters). A long unbraced length indicates that buckling would be the primary limit state.

<table>
<thead>
<tr>
<th>Test Run</th>
<th>Main Panel (με)</th>
<th>Support Strut (με)</th>
<th>Floor Beam (με)</th>
<th>Lateral Braces (με)</th>
</tr>
</thead>
<tbody>
<tr>
<td>001</td>
<td>50</td>
<td>10</td>
<td>32</td>
<td>23</td>
</tr>
<tr>
<td>003</td>
<td>45</td>
<td>7</td>
<td>27</td>
<td>23</td>
</tr>
<tr>
<td>004</td>
<td>30</td>
<td>6</td>
<td>23</td>
<td>23</td>
</tr>
<tr>
<td>005</td>
<td>30</td>
<td>6</td>
<td>18</td>
<td>23</td>
</tr>
<tr>
<td>007</td>
<td>20</td>
<td>3</td>
<td>8</td>
<td>30</td>
</tr>
<tr>
<td>008</td>
<td>25</td>
<td>5</td>
<td>7</td>
<td>33</td>
</tr>
<tr>
<td>009</td>
<td>25</td>
<td>6</td>
<td>8</td>
<td>22</td>
</tr>
<tr>
<td>011</td>
<td>25</td>
<td>6</td>
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<td>22</td>
</tr>
<tr>
<td>012</td>
<td>12</td>
<td>4</td>
<td>7</td>
<td>23</td>
</tr>
<tr>
<td>014</td>
<td>25</td>
<td>10</td>
<td>29</td>
<td>32</td>
</tr>
</tbody>
</table>

Table 14: Maximum strain for each test run.

The cross sectional area of the strut is 106.8 in² (689 cm²) and the weak axis moment of inertia is 5028.22 in⁴ (209,290 cm⁴). The critical buckling load can be calculated from (Gere & Goodno, 2009):

\[
P_{CR} = \frac{\pi^2 EI}{(KL)^2}
\]

Equation 5: Critical buckling load for a column.

Where:  
\( P_{CR} \) = Critical Buckling Stress  
\( E \) = Modulus of Elasticity; 29,000 ksi (200,000 MPa)  
\( I \) = Weak Axis Moment of Inertia; 5028.22 in⁴ (209,290 cm⁴)
K = Effective Length Factor; conservatively taken as 1

L = Length of Member; 846 in (21.5 meters)

From the above equation the critical buckling load is 2010.8 kips (559 kN). This would equate to 18.83 ksi (129.8 MPa) or 649 microstrain. The strain measured was small compared to this, although it did not take into account the dead load strains. However, it was determined that buckling should not occur.

3.4.3 Severity of Vibrations

The observed strains during live load testing were quite small in magnitude. The high frequency vibration cycles, while amounting to a virtually countless number of cycles, are at amplitudes of only a few microstrain. This would be equivalent to a stress level of less than 0.2 ksi (1.4 MPa). This low stress level is below the fatigue threshold for any structural detail as defined by the American Institute of Steel Construction(AISC, 2005) or by the American Association of State Highway Transportation Officials(AASHTO, 2004). Even the maximum pseudo-static strain amplitudes, occurring once or twice with each truck passage on the on-ramp, have amplitudes of around 50 microstrain or less, which is equivalent to a stress level of only 1.5 ksi (10.3 MPa). For the instrumented web panel the worst case detail, fatigue category E, has a stress threshold of 4.5 ksi (31.0 MPa)(AISC, 2005)(AASHTO, 2004). Even though the heaviest test vehicle utilized weighed only 48.3 kips (214.8 kN), any legal (80 kips (Sivakumar, Moses, Fu, & Ghosn, 2007)) or slightly overloaded truck on the on-ramp would still produce a pseudo-static response cycle below 125 microstrain or 3.6 ksi (24.8 MPa), which is below the fatigue thresholds for the details on the web panels.
The post buckling capacity of the girder can be calculated from the nominal shear strength due to tension field action as follows (AISC, 2005)(Salmon & Johnson, 1990).

From AISC Section G3.2 equation G3-2:

\[
V_n = 0.6 F_y A_w \left( C_v + \frac{1 - C_v}{1.15 \sqrt{1 + \left(\frac{a}{h}\right)^2}} \right)
\]

Equation 6: Post buckling capacity of a plate girder.

where:  
- \( F_y \) = yield strength of steel; 36 ksi (248 MPa)  
- \( A_w \) = area of the web; 45 in\(^2\) (290 cm\(^2\))  
- \( C_v \) = web shear coefficient  
- \( a \) = clear distance between transverse stiffeners; 72 in (1.82 m)  
- \( h \) = clear distance between flanges; 120 in (3.05 m)

The web shear coefficient can be calculated from Equation G2-5 (AISC, 2005):

\[
C_v = \frac{1.51 E k_v}{(h/t_w)^2 F_y}
\]

Equation 7: Web shear coefficient.

where:  
- \( E \) = modulus of elasticity of the steel; 29,000 ksi (200 GPa)  
- \( k_v \) = web plate buckling coefficient  
- \( t_w \) = thickness of the web; 0.375 in (9.5 mm)

and \( k_v \) can be calculated from

\[
k_v = 5 + \frac{5}{\left(\frac{a}{h}\right)^2}
\]

For the given dimensions the web plate buckling coefficient can be calculated as 18.9.

The web shear coefficient can then be calculated as 0.22. Using these numbers the shear
capacity of the web girder including tension field action is 779 kips (3465 kN). This is equivalent to a stress of 17 ksi (117 MPa). This is above any stresses encountered during testing in addition to the dead load stresses. It was also determined by Baker that the girder had adequate post buckling capacity (Michael Baker Jr. Inc., 2009).

3.5 Finite Element Modeling of Buckled Plate

Several attempts were made to predict the reduction in natural frequency in a buckled plate element and are presented in this section. The methods used involved modeling a plate with finite element software and varying the shear stresses in the member. The change in fundamental frequency as then determined. One of the methods utilized involved modeling the girder with the strip method, discussed below. The strip method analysis was performed and reported by: (Calderone, 2009). Another method that was attempted involved modeling the plate with anisotropic steel that had a reduced modulus of elasticity in the direction of the tensile strains in the girder web.

3.5.1 Strip Method to Model Behavior of Buckled Plate

The strip method involves modeling the plate with frame sections. The frames sections are placed at a 45 degree angle to the edges of the plate and distributed across the plate. Figure 54 shows a model of the plate using the strip method. The strips are oriented in the direction of principal stresses within the plate. Since the web of a plate girder is assumed to be in pure shear the principal stresses are oriented at 90 degrees on Mohr’s circle or 45 degrees on a stress block. Further, the magnitudes of the stresses are equal and the signs are opposite.
To determine the fundamental frequency a transverse load was placed at each node, intersection of two strips, of the model. The displacement of the center node was then recorded after performing a non-linear static P-Delta analysis. To simulate an increase in the shear stresses beyond buckling thermal strains were induced in the tension members of the model. The natural frequency of the system was then estimated from the following:

\[
k \approx \frac{1}{\Delta_y} \quad \text{and} \quad \omega \approx \sqrt{k}
\]

where: \( \Delta_y \) = maximum out-of-plane displacement of center node, in (mm)

\( k \) = out-of-plane stiffness, 1/in (1/mm)

\( \omega \) = fundamental frequency, Hz
The analysis determined a relationship between the applied shear stress, buckling stress, fundamental frequency of the unbuckled web, and the fundamental frequency of the buckled web. The equation to determine the buckled frequency is as follows:

\[
\omega' = \omega \sqrt[0.54864]{\frac{V}{V_{cr}}} - 1
\]

**Equation 8: Buckled frequency from strip method.**

where: \(\omega'\) = fundamental frequency at shear load “V”, Hz  
\(\omega\) = fundamental frequency of unbuckled plate  
\(V\) = shear load in web, kip (kN)  
\(V_{cr}\) = plate buckling shear force, kip (kN)

It was initially determined that the critical buckling stress in the web was approximately 5.5 ksi (37.9 MPa), Equation 3. The actual shear stress in the web was determined from the finite element model of the girder to be between 6 and 7 ksi (41.4 and 48.3 MPa). The first fundamental frequency of the unbuckled web was predicted to be 15 Hz from the unbuckled modal analysis. Equation 8 predicts the fundamental frequency of the buckled web to be between 8.2 and 14.3 Hz, depending on the value of the shear stress in the plate. The second fundamental frequency, 26 Hz for the unbuckled web, is predicted to be between 14.3 and 24.7 Hz. These results correlate well with the observed results from live load testing.
3.5.2 Anisotropic Steel Element to Model Behavior of Buckled Plate

The second method utilized to predict the fundamental frequency of a buckled plate involved using an anisotropic steel element. SAP2000 was used to model a plate element with a thickness of 0.375 inches. Initially a 10 foot by 10 foot plate was modeled. This was done to compare results with the previously mentioned analysis. The plate was supported on one corner with a pin, restraining displacements in the x, y, and z-directions. The other three corners were supported with rollers, restraining the displacements in the z-direction.

To ensure global stability, a node was offset from one corner and a link was added from that node to the nearest corner of the plate element. The offset node was modeled as a pin connection. The linkage bar restrained global rotations of the plate element. Heavy frame elements were also added around the edge of the plate to model knife edge supports. Figure 55 shows a screen shot of the model.

Figure 55: Finite element model of 10 ft by 10 ft thin plate.
The model was loaded with an arbitrary distributed load of 4.5 kips/ft along the edge to induce shear stresses in the plate, also shown in Figure 55. A linear static analysis was then performed to determine the von Mises effective stress. The effective stress in the plate element due to the shear load was then determined. The von Mises effective stress is used to compare different states of stress by converting all stresses to an effective stress that is comparable to an axially loaded specimen. To calculate the von Mises stress based on principal stresses the following relationship is used: (Dowling, 2007)

\[ \bar{\sigma} = \frac{1}{\sqrt{2}} \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2} \]

where: \( \bar{\sigma} \) = effective von Mises stress, ksi (kN)

\( \sigma_1; \sigma_2; \) and \( \sigma_3 \) are the principal stresses in the 1, 2, and 3 directions.

Figure 56 shows the von Mises stresses for the applied shear load. The average shear stress for a shear load with a multiplier of one is approximately equal to 3 ksi (20.7 MPa).
The fundamental frequency of the unbuckled system was determined to be 4.89 Hz from a modal analysis of the plate. The shift in fundamental frequency due to buckling and increased shear loads was done by first altering the modulus of elasticity of the material in the direction of principal compressive strains. This was accomplished by using an anisotropic steel and setting the modulus of elasticity in one principal direction equal to a much lower value. The plate was modeled with the anisotropic steel and the material angle was set to 45 degrees, aligning the principal direction with the principal compressive strains.

A nonlinear analysis was then performed on the model and the stiffness at the end of the analysis was used to perform a modal analysis. The applied shear load was then varied and the resulting fundamental frequency was computed. The ratio of the unbuckled to buckled fundamental frequency versus the effective stress was then plotted. Figure 57 shows the plots for several values of the modulus of elasticity in the compressive direction. Included in the plot is the equation of the trend line. It was determined that the model was ill-behaved if the modulus was less than 6000 ksi (41.4 GPa).

It can be observed that reducing the modulus reduced the initial ratio of fundamental frequencies; however, an increase in applied shear stress quickly increased the ratio. It was also noted that the relationship become more linear with a decrease in modulus. A relationship for the ratio of fundamental frequencies to applied shear stress can be determined for the lowest value of modulus of elasticity, E = 6000 ksi (41.4 GPa):
\[
\frac{\omega'}{\omega} = -6 \times 10^{-5} \tau^2 + 0.0109 \tau + 0.842
\]

where: \( \omega' \) = buckled fundamental frequency at the given shear stress, Hz  
\( \omega \) = unbuckled fundamental frequency, Hz  
\( \tau \) = applied shear stress above buckling stress, ksi (MPa)

Figure 57: Ratio of fundamental frequency versus effective stress.

An analysis was also performed to simulate the web of the plate girder. The above model was modified by reducing the width to six feet and reducing the height to eight feet, Figure 58. The analysis was then repeated as above for one value of modulus of elasticity, \( E = 6000 \) ksi (41.4 GPa). The analyses determined that the relationship between the ratio of the fundamental frequencies and the applied shear stress was approximately linear, Figure 59, as follows:
\[ \frac{\omega'}{\omega} = 0.0108\tau + 0.8648 \]

Equation 9: Ratio of frequencies from anisotropic steel model.

where: \( \omega' \) = buckled fundamental frequency at the given shear stress, Hz

\( \omega \) = unbuckled fundamental frequency, Hz

\( \tau \) = applied shear stress above buckling stress, ksi (MPa)

Figure 58: Finite element model of buckled web of plate girder.

If the critical buckling stress (5.5 ksi), predicted shear stress (6 to 7 ksi), and first fundamental frequency (15 Hz) are used in the above equation then a reduced fundamental frequency of 13.1 to 13.2 Hz is predicted, depending on the applied shear stress. The above equation predicts between 22.6 and 22.9 Hz for the second fundamental frequency. The first fundamental frequency that was predicted was somewhat higher than measured during testing, however, the second frequency agrees well with observed behavior.
3.5.3 Comparison of Results

Several methods were used to predict the fundamental frequencies of out-of-plane vibrations in the web of the plate girder. The first method consisted of a modal analysis of an unbuckled web. The second utilized the modified strip method and the third used an anisotropic steel element. Table 15 compares the results from each of the methods and the observed results from live load testing. The strip method appears to have predicted the observed data with the most accuracy; however the model was modified to match the observed behavior. Both models predict the behavior with reasonable accuracy, however, refinement of the methods are needed.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Predicted from unbuckled finite element model</th>
<th>Observed from live load testing</th>
<th>Predicted from strip method</th>
<th>Predicted from anisotropic steel method</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>15 Hz</td>
<td>11.5 Hz</td>
<td>11.5 Hz</td>
<td>13.1 Hz</td>
</tr>
<tr>
<td>2</td>
<td>26 Hz</td>
<td>22.7 Hz</td>
<td>19.5 Hz</td>
<td>22.8 Hz</td>
</tr>
</tbody>
</table>

Table 15: Comparison of out-of-plane frequencies.
3.6 Conclusions

An investigation into the nature and source of out-of-plane vibrations in the web of the widening girder adjacent to Pier 1 was conducted. The data suggests that the out-of-plane response is attributable to live loads applied to the widening girder, both by direct loads on the W 14th street entrance ramp and “seismic” loads from vibrations in the south truss being transmitted through the girder support elements. Since the web of the widening girder is assumed to be buckled under the dead load, any added live load tends to increase the amplitude of the buckled shape. Dynamic live loads tend to excite the web panels at the natural frequencies of the truss (3.5 Hz), the widening girder (3 Hz), and the two lowest natural frequencies of the out of plane vibrations (11.5 Hz and 22.7 Hz). The “visible” vibrations of the web panels appear to be primarily the 11.5 Hz natural frequency, which show up predominately in the “free vibration decay” following the passage of a heavy vehicle. Several attempts to model and predict the reduced natural frequencies were performed. The initial models predicted the natural frequencies with reasonable accuracy; however, more refined analysis is required.

It was determined that corrective measures were not required to control vibrations of the widening girder web panels, as the resulting vibrational response observed during testing would not be damaging. Out-of-plane response should be expected for a web panel as slender as this one, which is in a buckled state from the applied dead load. More than adequate post-buckling strength is available, however, and the vibrations would not appear to pose a fatigue concern at this time.
CHAPTER 4: EXPANSION JOINT OPENING

4.1 Introduction

The slope at the west bank of the Cuyahoga River Valley in the vicinity of the Innerbelt Bridge has had and continues to have stability issues. Two failure planes, one shallow and one deep, exist below the west end abutment and Pier 1. The shallow failure plane is located at approximately 40 feet (12.2 meters) below the surface. The slip rate of the shallow failure plane is approximately 0.06 inches per year (0.15 cm/yr). The deep failure plane is located at approximately 105 feet (32 meters) below the surface. In the vicinity of the bridge the slip rate of the deep failure plane is also approximately 0.06 inches per year (0.15 cm/yr) (BBC&M Engineering, Inc., 2005).

The bridge is designed to function as a cantilevered truss with suspended spans. An explanation of this structural system is as follows. The bridge is pinned at Pier 1, located on the west bank of the river, and a section of the truss extends to the west end abutment, as shown in Figure 60. At the west end abutment the bridge is supported by rocker bearings.

Figure 60: Slope movement at west end of bridge.

2 Note: much of the work presented in this chapter has been published elsewhere; see: (Cleary & Huckelbridge, 2010)
From Pier 1 the truss cantilevers out over the river completing unit 1. Starting at Pier 2, unit 3 cantilevers over the river to the west and continues to Pier 3 to the east. A suspended span, unit 2, is located between units 1 and 3. Unit 2 is connected to unit 1 by a link at the bottom chord of the truss and a deflection joint at the top chord of the truss. It is connected to unit 3 only by a link at the bottom chord, with a full expansion joint at the deck. Figure 61 shows a picture of the linkage.

![Figure 61: Picture of linkage at Units 2 and 3.](image)

The previously mentioned slope movement is causing the west end abutment and Pier 1 to move towards the east while Pier 2 remains fixed, since it is outside of the failure plane. Since there is a restraint at Pier 1, as the pier moves towards the east, Units 1 and 2 also move towards the east. This movement has caused the expansion joint located over the river to “close” and become inoperable. This could potentially cause large thermal stresses to develop, since the truss members cannot expand.
To open the expansion joint, allowing movement of the truss and reducing thermal stresses, units 1 and 2 were moved towards the west. The relocation was done in several steps. First the bridge was unbolted from Pier 1; since this is the only place units 1 or 2 have a horizontal restraint. The truss was then lifted at Pier 1 and lubricated sliding plates were inserted between the bridge bearing and the pier. The bridge was then lowered onto the sliding plates and jacks were inserted to move the bridge horizontally. Figure 62 shows a picture of Pier 1 with the hydraulic jacks for the vertical lift in place and the jacks for the horizontal displacements off to the side. After the bridge was moved to the predetermined location, the bridge was again lifted at Pier 1 and the sliding plates were removed. New anchor bolts were then grouted into holes cored into the pier and the truss was again pinned at Pier 1.

Figure 62: Hydraulic jacks at Pier 1.
4.2 Instrumentation

An instrumentation package was installed on the bridge during the relocation to aid in orchestrating the jacking operations and to ensure safety. Two locations were instrumented, truss members at the Span 2 expansion joint and at Pier 1. Figure 63 shows the location of the two sites.

A total of sixteen strain gauges, four displacement transducers, and four thermistors were located near the Span 2 expansion joint. The bottom chord truss members to the east (L300-L301) and west (L27-L28) of the linkage pin had four strain gauges each, at the inside and outside faces at the top and bottom of the members. The members to the west of the linkage pin also had two thermistors installed, on the inside and outside face. Displacement transducers were located at the top and bottom chords of the truss across the expansion joint. The north and south trusses had identical instrumentation packages installed. Figure 64 shows the layout of the instruments at the expansion joint (Huckelbridge & Cleary, 2009).
In addition to the instruments at the expansion joint, four tiltmeters and four thermistors were located at Pier 1, Figure 8. The north and south pier legs had tiltmeters and thermistors located to measure rotations in the east-west and north-south directions. Table 16 lists all of the instruments used during monitoring.

<table>
<thead>
<tr>
<th>Strain Gauges (16)</th>
<th>HITEC Model HBWF-35-125-6-20GP-NT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Displacement Transducers (4)</td>
<td>UniMeasure Model HX-PB-10</td>
</tr>
<tr>
<td>Thermistors (4)</td>
<td>Campbell Model 107</td>
</tr>
<tr>
<td>Tiltmeters/Thermistors (4)</td>
<td>Applied Geomechanics Tuff Tilt Model 801</td>
</tr>
<tr>
<td>Data Loggers</td>
<td>Campbell Model CR5000 (Pier 1)</td>
</tr>
<tr>
<td></td>
<td>Campbell Model CR9000 (Expansion Joint)</td>
</tr>
</tbody>
</table>

Table 16: Instruments used during monitoring.
4.3 Vertical Lift

In May of 2009, the bridge was lifted at Pier 1 to insert the sliding plates. The bridge was lifted by placing multiple hydraulic jacks under the north and south truss at Pier 1, Figure 65. The jacks had a capacity of 9,040 kips (40,200 kN) at each pier. The estimated vertical reactions at the piers was 5,004 kips (22,260 kN) at the north truss and 6,180 kips (27,490 kN) at the south truss. The south truss has a larger reaction due to the widened portion of the bridge on the south side.

![Figure 65: Kinematics of moving operations.](image)

The bridge was continuously monitored during the lifting operation. Figure 66 and Figure 67 show the expansion joint openings and truss member strains during jacking operations. The expansion joint at the top chord of the trusses “opened” approximately 0.4 inches (10.16 mm) when the bridge was lifted. The bottom chord of the truss moved approximately 0.08 in (2.03 mm) during this same event. When the bridge was lowered onto the sliding plates the expansion joint at the top of the truss line closed approximately 0.12 in (3.05 mm). The bottom line of the truss did not have a significant amount of movement when the bridge was lowered.
Figure 66: Expansion joint displacements during lifting operations.

Figure 67: Member strains during lifting operations.
All of the instrumented members went into tension during lifting operations. The L300-L301 members (east of the linkage pin) exhibited approximately 70 microstrain in tension. These members had approximately 50 microstrain of recovery when the truss was lowered. The L27-L28 members (west of the linkage pin) exhibited approximately 150 microstrain in tension and recovered approximately 100 microstrain.

Several observations were noted from the tilt meter data. The first is that the pier legs rotated towards each other during lifting operations and the second is that both pier legs rotated towards the east. The north pier leg rotated approximately 90 microradians towards the south and 200 microradians towards the east. With the assumed approximate height of the pier legs this rotation would correlate to a horizontal displacement of approximately 0.03 in (0.76 mm) and 0.07 in (1.83 mm) respectively. The south pier leg rotated approximately 90 microradians towards the north and 400 microradians towards the east. This would relate to approximately 0.03 in (0.76 mm) and 0.14 in (3.56 mm) respectively.

This behavior was expected since the expansion joint was closed. As the base of the pier continued to move towards the east, due to soil movement, the top of the pier was restrained. After removing the restraint at the top of the pier, the bolts connecting the pier to the truss, the top of the pier could move towards the east; removing any bending strains.

4.4 Expansion Joint Opening

To facilitate the horizontal movement of the bridge hydraulic jacks were placed in multiple locations and activated during stages. Table 17 shows the location and available
jacking force for all horizontal jacks. The total available horizontal jacking force was 6,720 kips (29,900 kN) and the total jacking force used during jacking operations was less than 1,400 kips (6,230 kN). The small percentage of the available jacking force used, less than 21 percent, is a testament to the effectiveness of the low friction sliding plate.

<table>
<thead>
<tr>
<th>Jack Location</th>
<th>Available Jacking Force kips (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>North Truss West End Pier</td>
<td>400 (1,780)</td>
</tr>
<tr>
<td>South Truss West End Pier</td>
<td>400 (1,780)</td>
</tr>
<tr>
<td>Widening Girder West End Pier</td>
<td>100 (445)</td>
</tr>
<tr>
<td>North Truss Pier 1</td>
<td>1,600 (7,120)</td>
</tr>
<tr>
<td>South Truss Pier 1</td>
<td>1,600 (7,120)</td>
</tr>
<tr>
<td>Span 2 Deck Expansion Joint</td>
<td>900 (4,000)</td>
</tr>
<tr>
<td>North Truss Upper Chord</td>
<td>400 (1,780)</td>
</tr>
<tr>
<td>Wind Shear Key</td>
<td>120 (535)</td>
</tr>
<tr>
<td>South Truss Upper Chord</td>
<td>400 (1,780)</td>
</tr>
<tr>
<td>North Truss Lower Chord</td>
<td>400 (1,780)</td>
</tr>
<tr>
<td>South Truss Lower Chord</td>
<td>400 (1,780)</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>6,720 (29,900)</strong></td>
</tr>
</tbody>
</table>

Table 17: Available horizontal jacking forces and locations.

During jacking operations the strains, displacements, and temperatures were continuously monitored at the Span 2 expansion joint. The power to the tiltmeters, however, was inadvertently shut down during this event, so there is only data from before and after the horizontal jacking. Figure 68 and Figure 69 show the expansion joint openings and truss member stresses during the horizontal jacking operations. Figure 70 shows the expansion joints located in Span 2 before and after the expansion joint opening.
Figure 68: Expansion joint openings during horizontal jacking.

Figure 69: Truss member stresses during horizontal jacking.
After jacking operations were completed the north and south trusses were moved approximately 2.5 in (6.35 cm) and 2.7 in (6.86 cm) respectively. Table 18 shows the final displacements at the four instrumented locations. The top chords of both trusses had a larger displacement, as expected, since there is less restraint at that location. The data also shows that the south truss was moved more than the north truss. This was done by design, since the south truss had moved more during slope movement, as determined by an on-going slope monitoring program.

<table>
<thead>
<tr>
<th>Transducer Location</th>
<th>Displacement in (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>North Truss Bottom Chord</td>
<td>2.35 (5.97)</td>
</tr>
<tr>
<td>North Truss Top Chord</td>
<td>2.49 (6.32)</td>
</tr>
<tr>
<td>South Truss Bottom Chord</td>
<td>2.67 (6.78)</td>
</tr>
<tr>
<td>South Truss Top Chord</td>
<td>2.77 (7.04)</td>
</tr>
</tbody>
</table>

Table 18: Truss displacements after horizontal jacking.

The strains in the instrumented truss members were low during jacking operations, as shown in Table 19. Member L27 – L28 on the south truss did, however, exceed the predetermined tensile limit of 150 microstrain by 10 microstrain. It was determined during jacking operations that it was safe to continue.
<table>
<thead>
<tr>
<th>Member</th>
<th>Tension Limit (με)</th>
<th>Compression Limit (με)</th>
<th>Net Change (με)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L300-L301 South</td>
<td>45</td>
<td>-25</td>
<td>-50</td>
</tr>
<tr>
<td>L27-L28 South</td>
<td>160 150</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>L300-L301 North</td>
<td>40</td>
<td>0</td>
<td>-180</td>
</tr>
<tr>
<td>L27-L28 North</td>
<td>105 200</td>
<td>None</td>
<td>None</td>
</tr>
</tbody>
</table>

Table 19: Maximum and net change in strain during horizontal jacking.

Figure 71 shows the tiltmeters up to and after horizontal jacking. The plot towards the top is the tiltmeter that measures the local east-west tilts on the north truss and the second one from the top measures the local east-west tilts on the south truss. The approximate displacement at the top of the foundation pier in the north pier is 0.04 in (1.02 mm) and 0.12 in (3.05 mm) in the south pier, both towards the west.
4.5 Expansion Joint Behavior

After the horizontal jacking of the bridge was completed, the bridge was continuously monitored for approximately 56 hours. Figure 72 shows a plot of the displacement transducers and thermistors immediately following the jacking operations. It can be seen that after the large thermal gradients, due to the sun shining directly on the truss members, had dissipated, the expansion joints operated nearly linear with respect to the temperature. Note that the temperature is plotted from lower to higher temperatures. Figure 73 shows a plot of the displacement transducers and thermistors for 56 hours following the horizontal jacking operations. It can be observed from Figure 72 and Figure 73 that as the temperature decreases the expansion joints “open” and vice-versa. These two plots clearly indicate that after jacking operations were completed, the expansion joints were working properly.
Figure 73: Expansion joint behavior following jacking operations.

4.6 Conclusions

The Innerbelt Bridge (CUY-90-15.24) needed rehabilitation to stay in service due to several structural issues. One major issue was the expansion joint in Span 2 that had closed due to slope movement at the west end of the bridge. The Ohio Department of Transportation determined that the expansion joint needed to be opened to extend the
bridges service life. To accomplish this task units 1 and 2, Span 1 and a portion of Span 2, were moved to the west.

The portion of the Innerbelt Bridge that was relocated was monitored throughout all jacking operations. Displacement transducers and strain gauges were monitored during critical stages to ensure safety. The results showed that strains/stresses were within reasonable limits and predetermined displacements were achieved. The required horizontal jacking forces, less than 1,400 kips (6,230 kN), was much less than the available jacking forces, 6,720 kips (29,900 kN). This indicates the efficiency of the low friction sliding plates that were used, as well as, the placement of horizontal jacks.

After the completion of all jacking operations it was determined that the expansion joint was properly functioning. This project outlines innovative ways to extend the service life of a structure and the importance of monitoring the structure to ensure safety and to verify the success of the project.
CHAPTER 5: WIDENING GIRDER REMOVAL

5.1 Introduction

In the early 1980’s the Innerbelt Bridge was modified to accommodate a West 14th street entrance ramp for eastbound traffic. To facilitate the entrance ramp a plate girder, shown in Figure 74, was added to the south side of the bridge at the west end abutment. The girder was used to widen the bridge for an acceleration lane. During the late fall and early winter of 2009 the widening girder was removed. Stresses in several members were monitored during the removal process. Note that this is the same member discussed in Chapter 3.

Figure 74: Picture of widening girder.

The girder removal process took place in several steps. Initially the bridge was closed to traffic and the steel sidewalk and crash barrier system were removed. Then temporary concrete jersey barriers were placed along the length of the bridge and the
bridge was reopened to traffic. The bridge deck was then removed and a new concrete parapet wall was installed prior to the removal of the widening girder. Figure 75 shows a schematic diagram of the parapet wall. To remove the girder a barge with a crane, Figure 76, was used. The crane would pick a section of the girder. Then the floor beams were removed and the section was cut off and lowered to the ground and cut up further.

Figure 75: Schematic diagram of concrete parapet wall (ODOT, 2009).
5.2 Instrumentation

Due to previous monitoring activities a total of 16 strain gages were located on four different members as shown in Figure 77. The strain gages used were HITEC model HBWF-35-125-6-20GP-NT and the data logger was a Campbell Model CR9000. These strain gages and the data logger were utilized to monitor member stresses during the removal of the widening girder.
5.3 Results

During the weekend closure of October 16th thru the 19th two events were observed. The first was an unloading of the structure that was observed in members L28 South and L300 South. The strain relief was approximately 27 and 28 microstrain (με) respectively. This unloading was assumed to be from the removal of portions of the walkway and railing system. The next event was a loading of the structure due to placement of temporary barriers. The observed event caused the following approximate strains: L300 North = 13 με, L28 North = 94 με, L300 South = 91 με, and L28 South = 60 με. Figure 78 and Figure 79 show the strains in the south and north trusses during the events.
Figure 78: Strains in south truss from 10/16/09 thru 10/19/09.

Figure 79: Strains in north truss from 10/16/09 thru 10/19/09.
The next observed event took place from 10/19/09 to 10/28/09 and was the removal of the concrete deck. A minimal amount of strain relief was observed in L300 North. In members L28 North and South the change in strain was approximately 84 and 15 microstrain, respectively. Member L300 South had approximately 10 microstrain of strain relief. Figure 80 and Figure 81 show the strains in the south and north trusses during the removal of the concrete deck.

Figure 80: Strains in L300 and L28 South during the removal of the concrete deck.
After the concrete deck was removed a new parapet wall was installed. An increase in strain was observed during this process. Members L300 North and South had an increase in compressive strains of approximately 10 and 15 microstrain and members L28 North and South had an increase of approximately 70 and 80 microstrain. Figure 82 and Figure 83 show the south and north trusses during this event.
Figure 82: Strains in the south truss during the placement of the parapet wall.

Figure 83: Strains in the north truss during the placement of the parapet wall.
During the removal of the temporary barriers a small amount of strain relief was observed as can be seen in Figure 84 and Figure 85. In members L28 North and South the change in strain was approximately 10 and 15 microstrain and in L300 South the change was approximately 12 microstrain. During the removal of the widening girder there was not any noticeable changes in any of the members as can be seen from Figure 86 and Figure 87.

Figure 84: Strains in the South Truss during the removal of the temporary barriers.
Figure 85: Strains in the North Truss during the removal of the temporary barriers.

Figure 86: Strains in the south truss during the removal of the widening girder.
5.4 Finite Element Modeling

An attempt to model the structure and replicate the loads from the removal of the widening girder was performed. A simple model of the bridge was first developed in SAP2000, Figure 88. Point loads were added to represent loads from the floor beam extensions. The loads due to the support struts in the bottom chord of the truss were also added. All girder loads were estimated from literature provided by Michael Baker Jr. Inc. A line load was also included that represented the concrete parapet wall that was added to the structure. The parapet wall load was determined from the volume of concrete per unit length, based on ODOT drawings.
A linear static analysis was performed and the results of each load, girder loads and parapet loads, were analyzed to determine the change in load in each instrumented member. The load was then divided by the cross-sectional area of each member to determine the average stress. The stresses were then divided by the modulus of elasticity, 29,500 ksi (203.4 MPa), to determine the average axial strain. Table 20 displays the results of the finite element analysis.

<table>
<thead>
<tr>
<th>Member</th>
<th>Girder Load kips (kN)</th>
<th>Parapet Wall Load kips (kN)</th>
<th>Difference kips (kN)</th>
<th>$\mu\varepsilon$</th>
</tr>
</thead>
<tbody>
<tr>
<td>L300-L301 N</td>
<td>2.7 (12)</td>
<td>-5.14 (-22.8)</td>
<td>-7.84 (35)</td>
<td>-2.74</td>
</tr>
<tr>
<td>L27-L28 N</td>
<td>0.08 (0.36)</td>
<td>0.01 (0.04)</td>
<td>-0.06 (-0.27)</td>
<td>-0.05</td>
</tr>
<tr>
<td>L300-L301 S</td>
<td>-195.7 (-871)</td>
<td>-53.65 (-239)</td>
<td>142.05 (632)</td>
<td>49.7</td>
</tr>
<tr>
<td>L27-L28 S</td>
<td>0.24 (1.1)</td>
<td>0.08 (0.36)</td>
<td>-0.16 (0.7)</td>
<td>-0.11</td>
</tr>
</tbody>
</table>

*Table 20: Finite element results for girder removal.*
The results indicate that both of the members in the north truss had additional compressive strains added to them due to the rehabilitation project. On the south truss the member to the east of the expansion joint, L300-L301, which is a compression member had a reduction in the compressive strains. Member L27-L28 on the south truss which is a tension member had a small amount of tensile strains removed. These results were expected since the removal of the girder was assumed to remove load from the instrumented members on the south truss and add a small amount of load to the members in the north truss.

5.5 Conclusions

The overall change in the strain for each member from the beginning to the end of the project was observed to be small for most of the members as can be seen from Figure 89 and Figure 90. Table 21 shows the overall change in strain for each of the members. It can be observed that most of the members had an increase in the compressive strains. If, however, we assume the same change in strain from adding and removing the temporary barriers (may or may not be accurate), then the change in strain is much smaller and two of the members have a slight amount of strain relief. If the overall strain gage readings are used (Figure 89 and Figure 90) the strain matches closely with what is observed from the aggregate of each event.

The results from the overall strain gauge readings and the assumption of equal strains from the addition and removal of the temporary barriers match the expected results. Since the members on the south side of the bridge should have a reduction in load, then member L300-L301, compression member, should experience tensile strains.
Member L27-L28 which is a tensile member should experience compressive strains. The members in the north truss should see an opposite result since these members should have a small increase in load due to the removal of the widening girder.

![Figure 89: Strains in the south truss from 10/16/09 thru 12/16/09.](image)

<table>
<thead>
<tr>
<th>Event</th>
<th>L300 North</th>
<th>L28 North</th>
<th>L300 South</th>
<th>L28 South</th>
</tr>
</thead>
<tbody>
<tr>
<td>Removal of Railing</td>
<td>-</td>
<td>-</td>
<td>28</td>
<td>27</td>
</tr>
<tr>
<td>Placement of Temporary Barriers</td>
<td>-13</td>
<td>-94</td>
<td>-11</td>
<td>-60</td>
</tr>
<tr>
<td>Removing of Deck</td>
<td>-</td>
<td>84</td>
<td>10</td>
<td>15</td>
</tr>
<tr>
<td>Pouring of Parapet Wall</td>
<td>-10</td>
<td>-70</td>
<td>-15</td>
<td>-80</td>
</tr>
<tr>
<td>Removal of Temporary Barriers</td>
<td>3</td>
<td>10</td>
<td>12</td>
<td>15</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>-20</td>
<td>-70</td>
<td>24</td>
<td>-83</td>
</tr>
<tr>
<td><strong>Total assuming Barrier addition and removal is equal</strong></td>
<td>-10</td>
<td>14</td>
<td>23</td>
<td>-38</td>
</tr>
<tr>
<td><strong>Total Observed (overall)</strong></td>
<td>-20</td>
<td>12</td>
<td>18</td>
<td>-61</td>
</tr>
<tr>
<td><strong>Results of FEM</strong></td>
<td>-2.7</td>
<td>-0.05</td>
<td>49.7</td>
<td>-0.11</td>
</tr>
</tbody>
</table>

*units of microstrain; positive implies strain relief*

Table 21: Changes in strain for each member during observed events.
Table 21 indicates that the results from the finite element model do not agree with the experimental results. The results do have the same sense, tension or compression, for most of the members but, the magnitude of the results is not the same. One contributing factor to the difference may have been the result of the strain gauges “drifting” over time. The strain gauges used are known to be reliable; however, the data collection period was over several months. Another possible cause of the discrepancy may have been the shift of ambient temperature over the course of the test. The test was started in the fall months and continued into the winter months. A shift to lower ambient temperatures would cause apparent compressive strains in the members. This phenomenon would explain the difference in strains from the experimental data and the finite element analysis. Since no temperature data was collected during the test, incorporating changes in the ambient
temperature is not straightforward. Local temperatures could be used to apply a correction factor to the model, however, this was not done.

It was determined from experimental and analytical results that the members in the south truss had a reduction in loads as a result of the project. The load reduction was small, however. A comparison of the reduction in strains due to the widening girder removal and strains from thermal expansions indicate negligible load reductions due to the removal of the widening girder.
CHAPTER 6: CONCLUSIONS

The Innerbelt Bridge is an aging interstate highway bridge, located in Cleveland Ohio, that has surpassed its design life. Due to economic reasons the bridges service life needed to be extended. Several investigations were conducted to determine the current state of the structure and to monitor the structure during structural repairs and modifications. A detailed finite element model of the bridge was developed by Michel Baker Jr. Inc and forensic investigations were conducted to validate the model. Investigations were also conducted to determine the severity of out-of-plane distortions in the web of a plate girder and to monitor the opening of an expansion joint located in Span 2. Finally, an investigation was conducted to monitor the removal of a portion of the bridge, to reduce dead load stresses.

It was determined that the computer model of the structure was overestimating the stresses in critical bridge members. Although this is a conservative result it was determined that the members could not carry the full legal load. Live load testing performed to validate the model indicated that strains/stresses were 39% of the predicted strains for member L717-L718 south, Span 8. For member L300-L301 south, Span 2, the response was 60% of the predicted response. It was also determined that the predicted curvature in the members, due to bending strains, were consistently higher then live load testing results. Live load testing determined that the structure may not be over loaded/stressed due to legal truck loads.

In the mid 1980’s the structure was widened by adding a plate girder to the south side of the bridge at the west end. During routine inspections it was observed that out-of-plane vibrations were occurring near the Pier 1 support, area of highest shear loads. An
investigation into the nature and severity of the vibrations was undertaken. It was determined that the web was buckled under dead loads and that tension field action was carrying the additional shear loads. The observed out-of-plane deformations were determined to be a result of an increase in the buckled shape while large vehicles were passing over the bridge. The investigation found that the dynamic stresses/strains produced were below the fatigue threshold and should not cause any concern. It was also determined that the pseudo static strains due to the passage of a truck were also below the fatigue threshold and were not considered “dangerous”.

Several attempts to model the widening girder and predict dynamic characteristics for the buckled web were performed. Two basic methods were investigated, the first using the strip method and the second an anisotropic steel element. Both models predicted a reduction in natural frequency with reasonable accuracy. The methods investigated need refinement to increase the accuracy of the results, however.

One major concern of the bridge is the slope stability failure that is occurring at the west end abutment and Pier 1. A failure plane located below the surface is causing soil creep that is forcing Pier 1 and the abutment to move towards the east. This in turn is causing the expansion joint in Span 2 to “close”. The inoperable expansion joint could cause high thermal stresses/strains in critical bridge members. The expansion joint was “opened” by moving units one and two approximately 2.5 inches towards the west.

During the span relocation an instrumentation package was installed. The monitoring included strain gauges, displacement transducers, thermistors, and tiltmeters. The bridge was continuously monitored during all stages of the span relocation. It was determined
that the stresses/strains were all within safe operating ranges during the relocation and that the expansion joint was successfully “opened”.

Officials from the Ohio Department of Transportation determined that the previously mentioned widening girder should be removed. During the removal processes strain gauges that were installed during the monitoring of the span relocation were utilized to monitor stresses/strains. The data was collected over several months and some information was lost due to power failures in the data collection system, however, data was collected during all major events. It was determined that the members were not overloaded at any point during the removal process. It was also determined that the strains in the instrumented members did not change significantly due to the removal of the widening girder, however, a small amount of strain relief was observed.

The above mentioned investigations were conducted to extend the service life and ensure the safety of the Innerbelt Bridge. The results of the research were used to determine and design needed improvements and modifications to extend the service life. It has been determined that the bridge is safe to keep in service until a replacement structure can be built.
CHAPTER 7: RECOMMENDATIONS FOR FUTURE RESEARCH

Although much of the research performed is unique to this structure, several of the studies are universal and should be extended. The verification of analytical models with testing is important for all structures. With the increased use of structural modeling programs, such as finite element analysis, verification of the results is imperative to insure a safe design. The model for this structure predicted higher stresses than encountered in the field; however this may not always be the case. If a model predicts lower stresses and is never verified, structural failure could occur. The results of the field testing performed for this project can not only be used to improve the model of this structure, but lessons learned can be incorporated into future models to improve their accuracy.

A review of the literature found few references regarding the dynamic behavior of buckled plates. It is desirable to predict the fundamental frequencies of a structure to reduce stresses due to dynamic amplifications. The analytical models presented were able to predict the shift in natural frequencies with reasonable accuracy; however, more accurate results are desirable. Continued research into analytical models and verification through controlled laboratory testing is recommended.
APPENDICES

Appendix A: Live Load Test from Chapter 2

A.1: Dynamic Load Test Results for Member L300-L301 South
A.2: Dynamic Load Test Results for Member L717-L718 South
Appendix B: Out-of-Plane Vibrations from Chapter 3
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