UNIVERSITY OF CINCINNATI

Date: 7-May-2010

I, Scott Lab, hereby submit this original work as part of the requirements for the degree of:

Master of Science

in Civil Engineering

It is entitled:

Structural Evaluation of LIC-310-0396 Box Beams with Advanced Strand Deterioration

Student Signature: Scott Lab

This work and its defense approved by:

Committee Chair: Richard Miller, PhD

Bahram Shahrooz, PhD

Eric Steinberg, PhD, PE

Richard Miller, PhD

Bahram Shahrooz, PhD

Eric Steinberg, PhD, PE
Structural Evaluation of LIC-310-0396 Box Beams with Advanced Strand Deterioration

Scott R. Lab
B.S. University of Toledo

Thesis submitted to the
Department of Civil and Environmental Engineering
College of Engineering
Division of Graduate Studies
University of Cincinnati

for partial fulfillment of the requirements for the degree of

Master of Science

June 2010

Committee:
Dr. Richard A. Miller, Ph.D., P.E.
Dr. Bahram M. Shahrooz, Ph.D., P.E.
Dr. Eric P. Steinberg, Ph.D., P.E.
Abstract

Four adjacent prestressed concrete box beams were removed from a decommissioned bridge and tested to destruction. The beams were 36 inches wide by 17 inches deep and had a span length of 37 feet 6 inches. Each beam contained fourteen, $\frac{1}{2}$" diameter, stress relieved prestressed strands. All of the prestressed strands were arranged in one layer, which was located below the stirrups. One beam was in good condition, with the only visible deterioration being longitudinal cracking. One beam had three corroded prestressed strands visible and a large spalled segment which, when removed, exposed two additional prestressed strands. The remaining two beams were badly deteriorated, with spalled concrete exposing seven corroded prestressed strands.

The provisions of the AASHTO LRFD Bridge Design Specifications, the AASHTO Standard Specifications, and the PCI Design Handbook were used to evaluate the loss of prestressing force. The measured prestressing force loss was closest to the AASHTO LRFD Bridge Design Specifications, but all three methods provided conservative estimates of prestressing force loss.

The bridge had been designed using the AASHTO Standard Specifications. The provisions of the Standard Specifications were conservative in determining the ultimate loading for the undeteriorated beam. The provisions of the Standard Specifications were also conservative in determining the ultimate load for the deteriorated beam if the
deteriorated strands were assumed ineffective. The deteriorated concrete box beams were all ductile and had final deflections exceeding L/50.

The original bridge was rated using the test results. When following the provisions of the AASHTO Standard Specifications, the designed service loading for the bridge would have exceeded the ultimate loading of one deteriorated middle span beam. A failure of this beam would not have caused a progressive failure of the middle span under service loadings. Following the Load Factor Rating method for the middle span beams, the loadings exceeded the strength of the members. Therefore, load restrictions would have been required for the bridge had it not been decommissioned.
Acknowledgements

I would like to thank my advisor, Dr. Richard Miller, for his support and assistance on this thesis project. I would also like to thank my thesis committee members Dr. Eric Steinberg and Dr. Bahram Shahrooz for their assistance.

The Ohio Department of Transportation (ODOT) funded this project. I would like to thank: Timothy Keller, Administrator, Office of Structural Engineering; Mike Loeffler, Assistant Administrator Bridge Operations Maintenance Section; and Monique Evans, former Administrator, Research and Development.

I would like to thank: Don Bosse and Prestress Services LLC, in Melbourne, Kentucky for storing the prestressed beams at their facility until they could be tested; Reis Concrete Products, Inc. for transporting the prestressed beams to the UC Center Hill Research Facility; Concrete Coring Company for drilling cores in each of the prestressed beams; and JZ Regional Contractor for removing and disposing of the tested prestressed beams. I would also like to thank David Breheim for his invaluable assistance at the lab.

I am grateful to the University of Cincinnati for providing me with a University Graduate Scholarship while I was receiving my Master’s Degree in Structural Engineering.
My fellow graduate students and friends deserve thanks for their support. Special thanks go to my family, whose love, support, and encouragement enabled me to finish this thesis project.
Dedication

I would like to dedicate this Master’s thesis

in loving memory of my father

Robert H. Lab, Jr., P.E.

3/13/52 – 1/10/09
## List of Figures

| Figure 1.1.1 | Typical Bridge Cross Section | 1 |
| Figure 2.2.1.1 | Lake View Drive Bridge Collapse | 6 |
| Figure 2.2.1.2 | Lake View Drive Beam Detail | 7 |
| Figure 3.1.1 | LIC-310-0396 Location (Google Map) | 17 |
| Figure 3.1.2 | LIC-310-0396 Middle Span (Southern Pier Shown) | 17 |
| Figure 3.1.3 | LIC-310-0396 Middle Span (Southern Pier Shown) | 18 |
| Figure 3.1.4 | LIC-310-0396 Middle Span (Northern Pier Shown) | 18 |
| Figure 3.1.5 | LIC-310-0396 South Span (Southern Abutment Shown) | 19 |
| Figure 3.2.1 | LIC-310-0396 Center Span Beam Detail | 20 |
| Figure 3.2.2 | Plan View of Beams Being Destructively Tested (14, 16, 17, and 19) | 21 |
| Figure 4.1.1 | Beam Orientation on LIC-310-0396 Bridge. Original bridge north is shown | 22 |
| Figure 4.1.2 | Beam Testing Layout | 23 |
| Figure 4.2.1 | Testing Frame (Beam 16 shown) | 24 |
| Figure 4.2.2 | Midspan Location | 25 |
| Figure 4.2.3 | Concrete Strain Gauge | 25 |
| Figure 4.2.4 | Clip Gauge | 26 |
| Figure 4.2.5 | Prestressed Strand Strain Gauge | 26 |
| Figure 4.2.6 | DCDT Sensor | 27 |
| Figure 4.2.7 | Wire Potentiometer | 28 |
| Figure 5.1 | Beam 16: East Face | 31 |
| Figure 5.2 | Beam 16: Expected Load vs. Displacement | 33 |
| Figure 5.5.1 | Beam 16: Load vs. Displacement before wire potentiometers detached | 41 |
| Figure 5.5.2 | Beam 16: Load vs. LVDT Displacement – Response-2000 | 42 |
| Figure 5.5.3 | Beam 16: Load vs. LVDT Displacement – AASHTO | 42 |
| Figure 5.5.4 | Beam 16: Load vs. Concrete Strain | 43 |
| Figure 5.5.5 | Beam 16: Load vs. Clip Strain | 43 |
| Figure 5.5.6 | Beam 16: Load vs. 1st Eastern Prestressed Strand Strain at 19’6.5” from the south end | 44 |
| Figure 6.1 | Beam 14: West Face | 45 |
| Figure 6.2 | Beam 14: Expected Load vs. Displacement | 47 |
| Figure 6.5.1 | Beam 14: Load vs. Displacement – Response-2000 | 54 |
| Figure 6.5.2 | Beam 14: Load vs. Displacement – AASHTO | 54 |
| Figure 6.5.3 | Beam 14: Load vs. Concrete Strain | 55 |
| Figure 6.5.4 | Beam 14: Load vs. Clip Strain | 55 |
| Figure 7.1 | Beam 17: West Face | 56 |
| Figure 7.2 | Beam 17: Expected Load vs. Displacement | 59 |
| Figure 7.5.1 | Beam 17: Load vs. Displacement – Response-2000 | 66 |
| Figure 7.5.2 | Beam 17: Load vs. Displacement – AASHTO | 66 |
| Figure 7.5.3 | Beam 17: Load vs. Concrete Strain | 67 |
| Figure 7.5.4 | Beam 17: Load vs. Clip Strain | 67 |
| Figure 8.1 | Beam 19: West Face | 68 |
Figure 8.2  – Beam 19: Expected Load vs. Displacement ........................................... 71
Figure 8.5.1  – Beam 19: Load vs. Displacement – Response-2000 ............................ 80
Figure 8.5.2  – Beam 19: Load vs. Displacement – AASHTO ........................................ 80
Figure 8.5.3  – Beam 19: Load vs. DCDT Displacement until its limit was reached ....... 81
Figure 8.5.4  – Beam 19: DCDT Displacement vs. Eastern Wire Potentiometer ............
Displacement until the DCDT sensor reached its limit ........................................... 81
Figure 8.5.5  – Beam 19: Load vs. Concrete Strain ...................................................... 82
Figure 8.5.6  – Beam 19: Load vs. Clip Strain ............................................................... 82
Figure 8.5.7  – Beam 19: Load vs. 2nd Western Prestressed Strand Strain at 9’8.25” .......
from the north end ................................................................................... 83
Figure 8.5.8  – Beam 19: Load vs. 3rd Western Prestressed Strand Strain at 10’3.5” .......
from the north end ................................................................................... 83
Figure 8.5.9  – Beam 19: Load vs. 4th Western Prestressed Strand Strain at 10’2.5” .......
from the north end ................................................................................... 83
Figure 8.5.10 – Beam 19: Load vs. 2nd Western Prestressed Strand Strain at 19’11.25” ...
from the north end ................................................................................... 84
Figure 8.5.11 – Beam 19: Load vs. 4th Western Prestressed Strand Strain at 19’9” .......
from the north end ................................................................................... 84
Figure 10.1.1 – Load vs. Displacement Beam Comparison ......................................... 95
Figure 10.1.2 – Load vs. Concrete Strain Comparison ................................................ 96

Appendix B – List of Figures

Appendix B Figure 5.1.1  – Beam 16: Broken prestressed strand along length of ..........
the beam .................................................................................................... 128
Appendix B Figure 5.1.2  – Beam 16: Broken prestressed strand hanging loose .......... 128
Appendix B Figure 5.1.3  – Beam 16: End prestressed strand hanging loose on the .......
east side ................................................................................................. 128
Appendix B Figure 5.1.4  – Beam 16: Exposed strands and stirrups at the northeast ......
end ........................................................................................................... 129
Appendix B Figure 5.1.5  – Beam 16: Two exposed corroded strands on the east .......
underside .............................................................................................. 129
Appendix B Figure 5.1.6  – Beam 16: Three exposed corroded strands on the east .......
underside .............................................................................................. 129
Appendix B Figure 5.1.7  – Beam 16: Four eastern exposed corroded strands .......... 130
Appendix B Figure 5.1.8  – Beam 16: Three eastern exposed corroded strands .......... 130
Appendix B Figure 5.1.9  – Beam 16: Three eastern exposed corroded strands .......... 130
Appendix B Figure 5.1.10 – Beam 16: Two western exposed corroded strands ......... 131
Appendix B Figure 5.1.11 – Beam 16: Three western exposed corroded strands ......... 131
Appendix B Figure 5.1.12 – Beam 16: Three western exposed corroded strands ......... 131
Appendix B Figure 5.1.13 – Beam 16: Two western exposed corroded strands ......... 132
Appendix B Figure 5.1.14 – Beam 16: Western strand cutoff point ......................... 132
Appendix B Figure 5.1.15 – Beam 16: Narrow undamaged section under the beam .... 132
Appendix B Figure 5.1.16 – Beam 16: Narrowest undamaged section under the ....... beam ................................................................. 133
Appendix B Figure 5.1.17 – Beam 16: Saw cut into the east side of the beam 133
Appendix B Figure 5.3.1 – Beam 16: Beam’s deflection during final loading cycle 134
Appendix B Figure 5.3.2 – Beam 16: Beam’s deflection during final loading cycle 134
Appendix B Figure 5.3.3 – Beam 16: Compression failure of top concrete 134
Appendix B Figure 5.4.1 – Beam 16: Transverse spalled concrete across entire beam underside 135
Appendix B Figure 5.4.2 – Beam 16: Underside damage after testing 135
Appendix B Figure 5.4.3 – Beam 16: Eastern five corroded strands 135
Appendix B Figure 5.4.4 – Beam 16: Western four corroded strands 136
Appendix B Figure 5.4.5 – Beam 16: Exposed stirrup above deteriorated strands 136
Appendix B Figure 6.1.1 – Beam 14: Longitudinal crack beside first eastern strand 138
Appendix B Figure 6.1.2 – Beam 14: Longitudinal crack under first eastern strand 138
Appendix B Figure 6.1.3 – Beam 14: Partial saw cut before readjusted outward at northwest end 138
Appendix B Figure 6.1.4 – Beam 14: Spalled concrete and stirrup rust at northeast end 139
Appendix B Figure 6.1.5 – Beam 14: Exposed corroded stirrups and strand on the southeast end 139
Appendix B Figure 6.1.6 – Beam 14: Corroded strand and rebar exposed at the southwest end 139
Appendix B Figure 6.3.1 – Beam 14: Cracks marked along the west side 140
Appendix B Figure 6.3.2 – Beam 14: Cracks marked along the east side 140
Appendix B Figure 6.3.3 – Beam 14: Corroded strand exposed after concrete was chipped off 140
Appendix B Figure 6.3.4 – Beam 14: Corroded strand with broken wires 141
Appendix B Figure 6.3.5 – Beam 14: Transverse cracking under beam 141
Appendix B Figure 6.3.6 – Beam 14: Top concrete failing in compression 141
Appendix B Figure 6.4.1 – Beam 14: West side damage from testing 142
Appendix B Figure 6.4.2 – Beam 14: West side damage after spalled concrete cleared 142
Appendix B Figure 6.4.3 – Beam 14: East side damage from testing 142
Appendix B Figure 6.4.4 – Beam 14: East side damage after spalled concrete cleared 143
Appendix B Figure 6.4.5 – Beam 14: Five western strands exposed without corrosion 143
Appendix B Figure 6.4.6 – Beam 14: Four eastern strands exposed with small corrosion patches 143
Appendix B Figure 6.4.7 – Beam 14: Cross section at the point of failure 144
Appendix B Figure 7.1.1 – Beam 17: West side longitudinal cracking 146
Appendix B Figure 7.1.2 – Beam 17: West side longitudinal cracking 146
Appendix B Figure 7.1.3 – Beam 17: West side longitudinal cracking 146
Appendix B Figure 7.1.4 – Beam 17: West side longitudinal cracking 146
Appendix B Figure 7.1.5 – Beam 17: Spalling prior to being removed 147
Appendix B Figure 7.1.6 – Beam 17: Four western corroded strands after spalling removed 147
Appendix B Figure 7.1.7 – Beam 17: Rust stains on west side 148
Appendix B Figure 7.1.8 – Beam 17: Spalling on the underside ................................. 148
Appendix B Figure 7.1.9 – Beam 17: Spalled concrete and corroded eastern strand... 148
Appendix B Figure 7.1.10 – Beam 17: Northeast beam corner with exposed ............
   longitudinal bar and prestressed strand ............................ 149
Appendix B Figure 7.3.1 – Beam 17: Beam deflecting during testing ..................... 150
Appendix B Figure 7.3.2 – Beam 17: West side cracking prior to failure ................. 150
Appendix B Figure 7.3.3 – Beam 17: East side cracking prior to failure ................. 150
Appendix B Figure 7.4.1 – Beam 17: Cracking going through the tie rod................. 151
Appendix B Figure 7.4.2 – Beam 17: East side damage from testing ...................... 151
Appendix B Figure 7.4.3 – Beam 17: Five eastern strands corroded ....................... 151
Appendix B Figure 7.4.4 – Beam 17: Five western strands corroded ..................... 152
Appendix B Figure 7.4.5 – Beam 17: Cross section at the tie rod point of failure..... 152
Appendix B Figure 8.1.1 – Beam 19: Exposed broken strand at southeast end of the......
   beam ................................................................................. 154
Appendix B Figure 8.1.2 – Beam 19: Spalled concrete and exposed strands on the......
   west side ........................................................................... 154
Appendix B Figure 8.1.3 – Beam 19: Three exposed corroded strands on the west......
   underside ........................................................................ 154
Appendix B Figure 8.1.4 – Beam 19: Four corroded strands exposed ...................... 155
Appendix B Figure 8.1.5 – Beam 19: After delaminated concrete removed, seven......
   corroded strands exposed ................................................. 155
Appendix B Figure 8.1.6 – Beam 19: After delaminated concrete removed, seven......
   corroded strands exposed ................................................. 155
Appendix B Figure 8.3.1 – Beam 19: Major crack opening under the southern load ......
   point ................................................................................. 156
Appendix B Figure 8.3.2 – Beam 19: East side damage under southern loading........
   point ................................................................................. 156
Appendix B Figure 8.3.3 – Beam 19: East side damage and DCDT sensor no longer .....
   connected to the guide beam ............................................. 156
Appendix B Figure 8.4.1 – Beam 19: Lateral buckling of the beam to the west...... 157
Appendix B Figure 8.4.2 – Beam 19: Three eastern exposed strands without..........
   deterioration ................................................................. 157
Appendix B Figure 8.4.3 – Beam 19: Western seven corroded strands ............... 158
Appendix B Figure 8.4.4 – Beam 19: Western four corroded strands and next three....
   good strands .................................................................... 158
Appendix B Figure 8.4.5 – Beam 19: Deteriorated stirrup above deteriorated..........
   strands .......................................................................... 158
Appendix B Figure 8.4.6 – Beam 19: Cross section at the southern loading point..... 159

Appendix C – List of Figures

Appendix C Figure 5.3.1(a) – Beam 16 East Side Cracks ................................. 162
Appendix C Figure 5.3.1(b) – Beam 16 East Side Cracks ................................. 163
Appendix C Figure 5.3.2(a) – Beam 16 West Side Cracks ................................. 164
Appendix C Figure 5.3.2(b) – Beam 16 West Side Cracks ................................. 165
Appendix C Figure 6.3.1(a) – Beam 14 East Side Cracks .............................................. 167
Appendix C Figure 6.3.1(b) – Beam 14 East Side Cracks .............................................. 168
Appendix C Figure 6.3.2(a) – Beam 14 West Side Cracks ............................................ 169
Appendix C Figure 6.3.2(b) – Beam 14 West Side Cracks ............................................ 170
Appendix C Figure 7.3.1(a) – Beam 17 East Side Cracks .............................................. 172
Appendix C Figure 7.3.1(b) – Beam 17 East Side Cracks .............................................. 173
Appendix C Figure 7.3.2(a) – Beam 17 West Side Cracks ............................................ 174
Appendix C Figure 7.3.2(b) – Beam 17 West Side Cracks ............................................ 175
Appendix C Figure 8.3.1(a) – Beam 19 East Side Cracks .............................................. 177
Appendix C Figure 8.3.1(b) – Beam 19 East Side Cracks .............................................. 178
Appendix C Figure 8.3.2(a) – Beam 19 West Side Cracks ............................................ 179
Appendix C Figure 8.3.2(b) – Beam 19 West Side Cracks ............................................ 180
List of Tables

Table 9.2.1 – Summary of Middle Span Load Rating Results ......................................... 91
Table 9.3.1 – Summary of Middle Span Capacity Results ............................................... 92
Table 10.1.1 – Summary of Deteriorated Prestressed Strands ...................................... 97
Table 10.1.2 – Summary of Destructive Testing Results ............................................. 98
Table 10.1.3 – Summary of Prestressed Strand Testing Results .................................. 99
Table 10.1.4 – Summary of Pre-Testing Response-2000 Behavioral Results ............ 100
Table 10.1.5 – Summary of Post-Testing Response-2000 Behavioral Results .......... 101
Table 10.1.6 – Summary of Ultimate Capacity Method Comparisons ....................... 102
Table 10.1.7 – Summary of Prestress Loss Results ..................................................... 103
Chapter 1

Introduction

1.1 – Introduction

Adjacent prestressed concrete box beam bridges are commonly found throughout the Eastern United States. These bridges have a favorable span to depth ratio, which helps when clearance is a problem. There are approximately 1,100 prestressed concrete box beam bridges in the Ohio Department of Transportation’s (ODOT) inventory. There are additional bridges of this type at the local level.

Prestressed concrete box beams are fabricated in a prestressing plant. After fabrication, the beams are placed side by side on bridge piers and pulled together with lateral tension rods. Grouted shear keys are used to connect the beams together. Sometimes, the lateral rods are post-tensioned to compress the shear keys, but older ODOT designs do not use lateral post-tensioning. If the bridge is non-composite, a waterproofing membrane and an asphalt wearing surface are placed overtop the beams. Composite box bridges have a composite concrete deck cast on top. A typical cross section of a prestressed concrete box beam bridge is shown in Figure 1.1.1 (MDOT 2005).

![Figure 1.1.1 – Typical Bridge Cross Section](image-url)
The main problem with adjacent boxes is that the shear keys crack and leak. Salt laden water can penetrate the concrete and begin to corrode the strand. Over time, the corroding prestressing strands cause the concrete to spall, exposing the strand. The strand may eventually rupture. Since the prestressed strands are enclosed by concrete, their deterioration level is hard to evaluate until the concrete spalls. When spalled concrete exposes corroded strands, their deterioration level can easily be determined, and often ruptured strands can be found hanging down from the beam when large amounts of concrete have spalled away. However, the prestressed strands can still be deteriorated and not be visible. This type of deterioration led to a bridge collapse in Pennsylvania (Harries 2006; Naito 2006). There have been structural failures of non-composite adjacent prestressed concrete box beam bridges in seven states (Macioce 2007). These states include Pennsylvania, Illinois, Ohio, Indiana, Florida, Colorado, and Virginia.

Since deterioration affects the beam’s strength, it is essential to understand how these deteriorated beams in service will perform. As the deterioration level of these strands increase the available prestressing force decreases, which could lead to additional cracking. If enough strands corrode, the strength of the beam could be compromised. By testing beams from a deteriorated bridge, more information can be gathered pertaining to deterioration of adjacent prestressed concrete box beams. The structural capacity and performance of similar deteriorated bridges still in service could then be better determined. Bridge inspectors could reference experimental tests for relevant knowledge when rating adjacent prestressed concrete box beam bridges.
This research project is part of a larger project being performed with Ohio University for the Ohio Department of Transportation (ODOT). Non-destructive testing is performed at Ohio University while destructive testing is performed at the University of Cincinnati.

1.2 – Objective

Bridges are inspected and rated visually. There are few studies which link the visual condition of the bridge to structural performance and a small number of tests which characterize the behavior of deteriorated beams. The objective of this study is to characterize the behavior of deteriorated box girders and link that behavior to the visual condition.

By performing this research, recommendations made to the Ohio Department of Transportation (ODOT) will aid bridge inspectors to best evaluate deteriorated adjacent prestressed concrete box beam bridges.
2.1 – Adjacent Prestressed Concrete Box Beams

Adjacent prestressed concrete box beam bridges are commonly found throughout the Midwestern portion of the United States. Other names for this bridge type include side-by-side and thin walled due to their alignment and dimensions respectively. Since the 1950’s, this bridge type has grown in popularity, and is still used today in various types of projects, from highway systems to local streets. According to Macioce (2007), the top five states with the most non-composite adjacent prestressed concrete box beam bridges are Illinois, Ohio, Indiana, Pennsylvania, and Florida. These five states alone have a total of over 17,000 bridges of this type at the state and local levels.

Prestressed box beams are precast elements which are fabricated off-site at a precasting plant. The concrete in these beams must cure before they can be transported to the construction site. Therefore, the prestressed box beams are built in advance so they are accessible when needed.

Once the bridge’s piers and abutments are built, the prestressed box beam portion of the bridge can be placed next to each other across the bridge width. Once they are in place, full depth shear keys are used to attach each box beam together. The shear keys allow there to be shear continuity between each of the beams. In some cases, transverse post tensioning is then added. For composite bridges, a bridge deck is then poured directly on
top of the box beams. Non-composite bridges usually have an asphalt overlay. If the structure is non-composite and does not have the lateral tie rods post-tensioned, the shear keys provide only lateral shear continuity. A composite deck and/or post-tensioned tie rods provide lateral flexural continuity as well.

There are many reasons to use adjacent prestressed concrete box beam bridges (MDOT 2005). This bridge type is easy to construct versus other bridges. Pouring a deck on top of the adjacent beams does not require extra formwork for support. Their span-to-depth ratio is an advantage when there are clearance issues. It is also economical to use adjacent prestressed box bridges. They have been proven to be durable and have a high torsional stiffness. Adjacent prestressed concrete box beam bridges can also be aesthetically pleasing and not detract from their surroundings.

2.2 – Related Research

There are many examples in literature of prestressed concrete box beams. Most of these tests have been conducted on box beams when there is either no deterioration or only minor deterioration. Only a few tests have been conducted on box beams containing major deterioration. The following presents some of these tests with various levels of deterioration.

2.2.1 – Lake View Drive Bridge Collapse

The Lake View Drive Bridge on State Route 1014 crosses over Interstate 70 in Washington, Pennsylvania. An exterior prestressed box beam failed and collapsed under
its own dead weight on December 27, 2005. The beam collapsed onto the highway below, as shown in Figure 2.2.1.1 (Harries 2009; Harries 2006; Naito 2006). Prior bridge inspections had shown this beam had major strand damage. The resulting investigation determined the damaged beam’s capacity was approximately equal to its own self-weight.

The bridge had four spans, each with eight adjacent box beams. The main span box beams were 42 inches deep by 48 inches wide. The beams had sixty, 3/8 inch diameter prestressed strands distributed across 5 rows. The Lake View Drive beam detail is shown in Figure 2.2.1.2 (Harries 2009).

![Figure 2.2.1.1 – Lake View Drive Bridge Collapse](image-url)
Since many similar bridges are in service, extensive studies were conducted on this bridge. The University of Pittsburgh performed destructive tests and Lehigh University performed forensic evaluations. Information from these studies is shown in the following sections.

2.2.1.1 – University of Pittsburgh Investigation

Full scale destructive testing was performed on two box girders from the Lake View Drive Bridge (Harries 2006; Harries 2009). The interior girder adjacent to the failed exterior girder was tested. The other exterior girder from the same span was also tested. Both of these girders were in better condition than the failed girder. The adjacent interior
girder had spalled concrete which had exposed twelve corroded strands, six of which were broken. The external girder also had corroded strands and damage initiated by vehicle impacts.

Testing was performed using two rock anchors applying line loads on the girders 48 inches apart. These loads were representative of an AASHTO tandem axle load. The interior girder failed suddenly with prestressed strands rupturing. This interior girder was in a considerably worse state than determined by its pre-testing assessment. Testing was stopped short on the external girder, because of large lateral instability and a perceived brittle failure.

Harries made these recommendations to the Pennsylvania Department of Transportation:

- Reduced concrete cover results in a higher likelihood of strand damage occurring.
- Unexposed strands adjacent to visibly damaged strands can be damaged. Chipping away unsound concrete enables the state of strand damage to be evaluated.
- Corroded and ruptured strands should be disregarded when counting their contribution to the beam. To be conservative, redevelopment of these strands away from the damaged sections can be neglected.
  - The total number of deteriorated strands in a damaged section should be increased by 25% to account for non-visible corroded strands.
o When there are multiple layers of strands, the total number of deteriorated strands in a damaged section should be increased by 25% to account for damage to second layer strands.

- Shear keys should be ignored when load rating adjacent box beam bridges. Water damage between beams is evidence of their damage.
- Camber can be evaluated to determine the approximate amount of prestressing force available.

### 2.2.1.2 – Lehigh University Investigation

Forensic evaluation of beams from the Lake View Drive Bridge was conducted by Lehigh University (Naito 2006). Eleven beams were investigated from this bridge. The main focus of the study was on two exterior and one interior beams.

Naito’s forensic findings include the following:

- The concrete compressive strength exceeded requirements. The concrete quality met specifications with the entrained air, aggregates, and cement.
- The prestressing strand tensile strength exceeded requirements.
- The minimum bottom flange thickness, the minimum web thickness, and the bottom strand clear cover distance was found smaller than in the design.
- The bottom prestressed strand layer was not designed to be contained by the stirrups. This was a common practice when the bridge was designed. Now the stirrups are designed to be below the bottom strand layer.
• The cardboard beam void forms had shifted during beam production, evident by inconsistent web and flange thickness measurements. Cardboard void forms are no longer used.

• Top vent holes were installed during fabrication into each void region and were made of the same material as the bottom drain holes. The top vent holes were not present on the drawings and allowed surface water to drain into the beam voids. One beam void was half filled with water. The water in the voids had elevated chloride concentrations, leading to corrosion from within the beam.

• High chloride levels were found on the bottom flanges and interior webs. These levels decreased as the penetration distance increased.

The conclusions from the forensic investigation include the following:

• Top vent holes should be sealed while the drainage hole should be increased.

• The strands above a longitudinal crack and at least one strand on either side should be ignored when calculating the beams strength.

• Hairline cracks on the beam bottom could indicate corrosion on the above strand.

• Helical strand wires allow the possibility all of the exterior wires to be exposed to corrosion over a short longitudinal length.

2.2.2 – Parekh and Miller

Destructive tests were conducted by Parekh and Miller on both a 12 year old deteriorated prestressed box beam and an undamaged control beam (Miller 1994). The damaged beam was 36 inches wide, 33 inches deep, 76.5 feet long, and 5 inch thick walls. The
beam had 18 tendons, of which 16 were in the bottom layer and 2 were in the second layer. The prestressing strands were 0.5” diameter grade 270 strands. Since no undamaged beam existed for use as a control beam, one was cast with similar properties.

The deteriorated beam had one broken tendon and two corroded tendons out of its 18 tendons. These were located in a corner of the beam. The beams were tested with point loads that were 21 feet apart centered along the beam.

From their destructive testing, Parekh and Miller found:

- The asymmetry of the beam, caused by deterioration, contributed to the lateral instability and collapse.
- The 1989 AASHTO Standard Specifications were conservative in determining the cracking moment while they were not conservative in determining the ultimate moment.
- The steel had yielded just before the deteriorated beam failed.
- Web cracks in the deteriorated beam extended farther into the web and were located farther apart due to the lack of prestressing steel in the sections.
- The undamaged beam did not fail while the damaged beam had a sudden failure.

2.2.3 – Shenoy and Frantz

Shenoy and Frantz performed tests on two 27 year old prestressed concrete box beams removed from a deteriorated bridge (Shenoy 1991). This bridge was rated 4 out of a possible 9, with 0 being critical and 9 being new condition. The deterioration of these
box beams consisted of staining due to water seepage and minor cracking and spalling. These beam’s dimensions were 36 inches wide, 27 inches deep, and 54 feet long. There were 22 – 7/16” diameter, Grade 250 strands in each beam, of which 7 were in the second prestressing layer. The type of strand was not known, but they were assumed to be stress relieved strands. The box beams were tested with point loads applied at third points along the beams.

From their test results, Shenoy and Frantz were able to conclude:

- The prestress losses were approximately half of what was expected using the PCI method.
- The flexural strength exceeded the required strength at factored loads using the 1989 AASHTO Standard Specifications.
- The maximum strand stresses were similar to what was expected using the ACI 318-89 Code.
- The load-deflection curve behavior was as predicted. There was elastic behavior prior to flexural cracking followed by the curve flattening as it approached the ultimate capacity.
- The beams were ductile and behaved as anticipated.

2.2.4 – Naito, Sause, and Thompson

Naito, Sause, and Thompson performed tests on two 12 year old prestressed concrete box beams (Naito 2008). The beams came from a three-span continuous bridge with a cast-in-place deck. The beams were four foot square precast prestressed box beams. The
beams had 30 strands in span 1, 36 strands in span 2, and 20 strands in span 3. The beams had 0.5” diameter grade 270 low relaxation strands. These beams were found to have cracks near the piers, abutments, and the transition from solid to box regions of the beams. These cracks were unexpected due to the amount of prestressing in these portions of the beams. The bridge was replaced and an investigation was conducted. The beams were tested twice with pin and roller supports in three-point bending until the load capacity decreased. For the first test, the loading was 11 feet 8 inches from the abutment end of the beam. The second test configuration had the damaged region cantilevered past the support, and the loading applied 11 feet 8 inches from the pier end of the beam. The flexural failure of the beam was caused by strand failure.

Naito, Sause, and Thompson developed the following conclusions from their tests:

- The prestress loss was greater than expected by the PCI and AASHTO methods.
- Thermal and creep demands can open the existing cracks in the beams.
- The cracks formed in a discontinuity region not expected by beam theory calculations.

Naito, Sause, and Thompson presented the following recommendations for the design and construction of prestressed concrete box beam bridges:

- The solid region of the box beams should be larger than the prestressed strands transfer length.
- Prestressing strands should not be exposed to contaminants which reduces the bond strength with the concrete.
• Positive moment continuity reinforcement should extend into the prestressed box beam at least the development length of the reinforcement bars plus the transfer length of the prestressing strands.

• Positive moment continuity reinforcement should have staggered cut-offs and be positioned inside the beam stirrups.

2.2.5 – Labia, Saiidi, and Douglas

Two 20 year old pretensioned box girders were tested by Labia, Saiidi, and Douglas (Labia 1997). The girders were approximately 70 feet long. Each girder had thirty, 0.5” diameter grade 270 stress relieved strands. The bridge they were taken from was demolished due to problems with cast-in-place components, not the precast beams. Testing was conducted using two loading points located 6 feet apart centered along the beam. Both girders experienced a brittle failure due to top compression.

Based on their results, Labia, Saiidi, and Douglas determined:

• The actual prestress loss was significantly higher than predicted by using the ACI method. Large temperature and humidity ranges might have attributed to this loss.

• The ultimate load obtained was in agreement with the ACI method.

• The unexpected brittle failure was attributed to a low amount of prestress and a significantly lower concrete ultimate strain than expected.
2.2.6 – Research Summary

When there is deterioration on a prestressed concrete box beam bridge, determining to what extent this deterioration has affected the bridge’s capacity is not straightforward. Visible prestressed strands which are corroded can be discounted when determining a beam’s capacity. However, the adjacent strands can also be deteriorated even though they are not visible to be evaluated. Cracking is typically an indicator of hidden deterioration to prestressed strands. Mistakes during fabrication can cause deterioration to progress, especially if the clear cover is less than designed. Previous studies have shown that there are usually additional deteriorated prestressed strands adjacent to any visible deteriorated strands. However, there is no set rule to follow for the number of additional prestressed strands to discount when calculating a beam’s capacity.

The method being used (AASHTO, ACI, or PCI) can affect the amount of prestress loss expected in a box beam. However, when comparing previous studies using the same methods, the amount of prestress loss was higher than expected for some beams, while for others it was lower. The previous studies suggest that the provisions of the AASHTO Standard Specifications and/or LRFD Specifications are conservative when applied to beams with little or no damage. The validity of applying these provisions to deteriorated beams seems to be dependent on an accurate assessment of the level of deterioration in the beam, thus demonstrating the need for the current study.
3.1 – Bridge Background

The beams tested for this investigation originally came from the LIC-310-0396 Bridge\(^1\). Located south of Pataskala, Ohio (Figure 3.1.1) in Licking County, the bridge went into service in 1980. The bridge consisted of three spans measuring 31’4”, 37’6”, and 31’4” from center to center of bearing. Each span consisted of 12, three-foot wide, adjacent prestressed concrete box beams. The original asphalt overlay was replaced in 1999. At the same time, the contractor removed and replaced the shear key mortar with a mortar including an epoxy resin. At the abutments, a polymer modified asphalt expansion joint system was placed.

At the time the bridge was taken out of service, the level of deterioration varied among the beams. Some beams had little or no deterioration. Other beams had major deterioration, with spalled concrete exposing corroded prestressed strands. Some of these corroded strands were broken and hanging down from the bridge. Figures 3.1.2 through 3.1.5 show the bridge’s state of deterioration during June of 2007, with the beams labeled according to their beam number on the bridge plans (Appendix A). The beams were removed from the bridge on July 17, 2007.

---

\(^1\) Note: ODOT nomenclature for bridge naming is as follows: The first three letters are an abbreviation for the county in which the bridge is located. The first set of numbers is the route number of the road. The second set of numbers is the mile marker measured from the south or west county line.
Figure 3.1.1 – LIC-310-0396 Location (Google Map)

Figure 3.1.2 – LIC-310-0396 Middle Span (Southern Pier Shown)
Figure 3.1.3 – LIC-310-0396 Middle Span (Southern Pier Shown)

Figure 3.1.4 – LIC-310-0396 Middle Span (Northern Pier Shown)
3.2 – Beam Descriptions

Each of the prestressed concrete box beams from the LIC-310-0396 Bridge were 17 inches deep by 36 inches wide with a 25° right forward skew (bridge drawings shown in Appendix A). These beams were fabricated according to applicable ODOT standard details at the time. These standards used a cover of 1.5 inches to the strand and the stirrups were placed above the strands. This standard is no longer used.

The prestress strands in the beams were ASTM A416 ½ inch 7-wire, uncoated, Grade 270 stress relieved strands. The initial design tension was 28,900 pounds per strand. Beams in the middle span contained 14 strands, while beams from both end spans contained 8 strands (refer to bridge drawings in Appendix A). Stirrups in the bottom of the beam were spaced at 24 inches except at the ends, where they were spaced at 6
Stirrups in the top flange were spaced at 12 inches. Four #5 longitudinal steel reinforcing bars were placed at the top of the beam and inside of the stirrups.

Steel tie rods were used to attach the adjacent box beams together at mid-span, but these were not post-tensioned. The lateral rods were simply hand tightened to pull the beams together. The center of the box beam was hollow, with the void space formed with Styrofoam™. This Styrofoam™ was used between the beam ends and the tie rods, where full depth concrete diaphragms were utilized. The beam detail for a center span beam can be found in Figure 3.2.1.

Each of the beams being tested in this investigation came from the center span. They are numbered 14, 16, 17, and 19 in Figures 3.1.2 to 3.1.4. The bridge’s plan view with the beam numbers marked is shown in Figure 3.2.2.

![Figure 3.2.1 – LIC-310-0396 Center Span Beam Detail](image-url)
Figure 3.2.2 – Plan View of Beams Being Destructively Tested (14, 16, 17, and 19)
Chapter 4

Testing Procedure

4.1 – Testing Setup

The beams were tested at the UC Large Scale Test Facility located at the Center Hill Research Center in Cincinnati, Ohio. The testing setup conditions for each beam were kept as identical as possible among all four beams being tested. The first task was locating the beam’s centerline and marking it on the beam. The beam was then positioned with its centerline directly under the center of the testing frame. The beam orientation being used throughout this research project was based on the beam orientation on the LIC-310-0396 Bridge. While the true north direction was slightly skewed from the beam orientation on the bridge, the northernmost end was referred to as the north end throughout this project for simplicity, as shown in Figure 4.1.1. Based on the orientation from the bridge, the beam spanned from north to south in the testing frame. The supports were then placed under the north and south ends. Bearing pads were used between the beam and the supports.

Figure 4.1.1 – Beam Orientation on LIC-310-0396 Bridge. Original bridge north is shown
The beams were loaded in four-point bending, with two loading points and two reaction points. The load from the testing frame was applied at two loading points which were located 6’0” apart and centered on midspan of the beam (Figure 4.1.2). Bearing pads were used at the loading points.

The distance to the supports was measured from center to center of bearing. There was damage at the ends of some of the beams due to deterioration and/or the removal process. As a result, it was not possible to get good bearing at the end of some beams. For the worst case (beam 16), the maximum usable span length was 33’6”. Therefore, all of the remaining beams were tested with 33’6” spans so the results were comparable. This “as tested” span was shorter than the 37’6” center to center of bearing in the actual bridge. All analytical results are based on the test span length, not the in-service span length.

![Figure 4.1.2 – Beam Testing Layout](image)

4.2 – Instrumentation

The testing frame had a 220,000 pound capacity (Figure 4.2.1). The available actuator stroke was 12 inches. Load was applied from the actuator at one point onto a steel loading beam. This load was then applied at two loading points 6 feet apart and centered
symmetrically about the midspan of the beam. The actuator movement was measured by a linear variable differential transformer (LVDT).

**Figure 4.2.1** – Testing Frame (Beam 16 shown)

Electrical resistance strain gauges were used to measure the strain in the concrete. The strain gauges were attached to the top side of the beams at midspan near the edges. Since
this was a skewed beam, the midspan was measured along the longitudinal centerline of the beam and a line was drawn perpendicular to the longitudinal centerline, as shown in Figure 4.2.2. The gages were placed near the edges of the beam along the perpendicular line. Figure 4.2.3 shows a strain gage.

![Figure 4.2.2 – Midspan Location](image.png)

After the beam cracked, clip gauges were placed across cracks to measure the strain. The data obtained were used to determine the estimated effective prestressing force in the beam. Supplemental calculations are shown in Appendix E3 for finding the effective prestressing force for each beam. A clip gauge is shown attached across a crack in Figure 4.2.4.

![Figure 4.2.3 – Concrete Strain Gauge](image.png)
Electrical resistance strain gauges were attached to prestressed strands when possible. The prestressed strand deterioration level combined with the strain gauge’s small size made attaching these gauges to the strands on the underside of the beams difficult, so only some exposed strands were gauged (beams 16 and 19). A strain gauge is shown in Figure 4.2.5 attached to a prestressed strand wire.

For the last beam tested (beam 19), a direct current differential transformer (DCDT) sensor was used to measure the lateral displacement during testing at midspan. This sensor is shown in Figure 4.2.6.
Celesco wire potentiometers with 15-inch capacity were used to measure the beam’s vertical deflection during testing. The wire potentiometer was firmly attached to both the beam at the midspan and on the ground as shown in Figure 4.2.7. Wire potentiometers were used on both sides of the beam at midspan because unsymmetrical deterioration could lead to vertical deflections varying along the beam’s width.
MTS Test Star software was used to control the load applied by the testing frame during testing. The controller was an MTS closed loop which was operated in position control. The beams were loaded at a rate of approximately four loading increments per 0.1 inches. After softening occurred, the beam loading increment was either 0.05 inches or 0.1 inches.
4.3 – Testing Procedure

The beams were tested in displacement control using an approximate increment of 0.03” of actuator displacement for each loading step. This increment was increased as the beam softened after cracking to a maximum of 0.1” of actuator displacement. A small initial test load was placed on the beams to insure proper functioning of the gauges. Then the actual testing began with load being applied until cracking occurred. Load was applied and then held steady while searching for cracks. If cracks were found, they were marked. Otherwise, more loading was applied and the process repeated.

After cracking occurred, the beam was unloaded and clip gauges were placed across these cracks. The locations of these clip gauges were measured and recorded. Once they were attached, load was reapplied to the beam until the cracks grew again. When the cracks opened up, the graph of the strain across the crack changed slope. This slope change enabled the effective prestressing force to be calculated. This process is explained and shown for each beam in Appendix E3. After the clip gauges had performed their purpose, the beam was unloaded and the gauges were taken off the beam to avoid being damaged.

The beam was then reloaded as before. Testing paused while new cracks were marked. Loading was continually applied in this manner until the beam failed. If the testing actuator ran out of stoke, the beam was unloaded and more bearing pads added at the load points. Testing then resumed with additional available stroke.
Near the end of testing, failure could be reached by meeting different criteria. If any of the following were achieved, the beam was considered to have failed:

- When the beam collapsed.
- When the beam’s deflection exceeded one fiftieth of the span length (8 inches).
- When the load exceeded the beam’s capacity by at least fifteen percent.

The test was not necessarily stopped at “failure”, but once the beam reached failure the test could be considered valid and complete.

After testing was completed, the cracking in the beam was documented and concrete cores were taken. This enabled the average concrete core strength to be determined. The concrete cores were tested by Ohio University according to AASHTO T24-02 / ASTM C42-99.
Chapter 5
Beam 16

5.1 – Beam 16: Pre-Testing Assessment

The first test was performed on beam number 16. This beam’s location was the fourth beam inward on the western side of the bridge’s center span, as shown in Figures 3.1.2 to 3.1.4. This beam can be seen on the bridge’s plan view as shown in Figure 3.2.2. In the following description of the beam, all directions correspond to the beam’s orientation on the bridge, which is shown in Figure 4.1.1. The beam was tested in a north and south orientation. Figure 5.1 shows the east face of the beam.

![Figure 5.1 – Beam 16: East Face](image)

Beam 16 had major deterioration along the entire beam length. There were many areas containing spalled concrete and exposed strands. The most significant deterioration corresponded to the first prestressed strand on the western side of the beam. The concrete cover had spalled off and exposed this strand over a length of 17’4”. This strand had broken and was hanging loose. Appendix B Figures 5.1.1 and 5.1.2 show this strand hanging down. The first strand on the east side was also hanging down at the southern end, as visible in Appendix B Figure 5.1.3. On the east side at the northern end, spalled
concrete had exposed two corroded stirrups and prestressed strands on the side of the beam (Appendix B Figure 5.1.4). The initial camber of the beam was 2”.

The amount of deterioration on the underside of the beam varied along the length from no visible deterioration to major deterioration with corroded strands visible. Exposed corroded strands were visible in groups ranging from one strand up to four strands wide at various spots along the beam. Appendix B Figures 5.1.5 through 5.1.16 show these areas of exposed prestressed strands and their respective locations on the beam.

When the beams were cut apart to be saved for testing, the saw cut into the east side of the beam, accidentally narrowing the bottom width by 2” (Appendix B Figure 5.1.17). Vertical hairline cracks were present on the beam sides at the top. Some of these cracks were on the beam and others on the grouting material. Some continued onto the top of the beam.

A total of seven corroded prestressed strands were found by visual inspection before testing beam 16. The specified concrete strength for the beam was 5,500 psi, but testing performed at Ohio University on other beams from this bridge showed strengths anywhere from 8,000 to 10,000 psi. Based on tests, an assumed concrete strength of 8,000 psi was used (Gulistani 2010). From calculating the loss of prestressing force from the provisions of the AASHTO LRFD Bridge Design Specifications, a value of 5.3 micro-strain was used for the effective strand prestressing-strain (calculations shown in Appendix E1.1). The beam was modeled in Response-2000 (Bentz 2000). The dead
weight of the beam was removed from the results. This allowed the live loading from testing to be compared against the theoretical results. With seven of the fourteen strands assumed effective, the expected loading was 47.0 kips total or 23.5 kips/point and the deflection would be 18.3". Following the method suggested by Naito (2006) of assuming strands adjacent to deteriorated strands are also deteriorated, one additional strand would be discounted from each side of the beam. Therefore, with five strands assumed effective the expected loading would be 33.1 kips (16.6 kips/point) and the deflection would be 18.5". The loading versus displacement curves from Response-2000 for both of these cases is shown in Figure 5.2.

![Figure 5.2 – Beam 16: Expected Load vs. Displacement](image-url)
5.2 – Beam 16: Testing Setup

The largest span length possible for testing this beam was 37’6”. This was shorter than the span length on the bridge. Damage in the bearing area caused the supports to be located farther inward, thus shortening the testing span length. The north support was 13’9” inches from the center of the load point to the center of bearing. The south support was 13’8” inches from the center of the load point to the center of bearing. The testing layout can be seen in Figure 4.1.2. The chosen dimensions enabled the supports to be located at sound concrete. The supports used in testing beam 16 consisted of a concrete filled steel tube, a bearing plate, and a bearing pad. The total height of each support was 1’0”.

Two concrete strain gauges were placed across the top centerline near the east and west edges. Wire potentiometers were attached to both the east and west sides at the center of the beam span. A strand strain gauge was placed on the center wire on the first prestressing strand on the eastern side of the beam. This strand was badly deteriorated. Of the seven helically wrapped wires in the strand, only the center wire and one wire wrapped around it were still intact.

5.3 – Beam 16: Destructive Testing

Beam 16 was tested on March 6th, 2009. Cracking was first visible on both sides of the beam at an applied load of 32 kips (16 kips/point). After cracking occurred, clip gauges were attached across a crack on both sides of the beam. The clip gauge on the west side was 14’3” from the north support and 1’1.25” down from the top of the beam. The clip
gauge on the east side was 15’3” inches from the north support and 1’0.25” down from the top of the beam.

During the test as load was applied, loud “popping” sounds were heard as the prestressed wires broke. At approximately 38 kips (19 kips/point), the load dropped suddenly as a strand broke. It was not possible to determine which strand broke from a visual inspection. Strand wires continued to break during the test, as causing the total load to remain in the range of 32 and 35 kips. Concrete cover spalled off throughout the test.

The actuator ran out of stroke several times during testing. Each time the stroke was exhausted, the beam was unloaded and another set of bearing pads was added at the loading points. For the final loading cycle, a 4” by 4” timber beam was added to each load point since all the available bearing pads were currently in use.

Testing concluded when the beam reached the floor and once again the actuator ran out of stroke. Appendix B Figures 5.3.1 and 5.3.2 show the beam during the final loading cycle when it deflected to the floor. With the beam’s deflection reaching the floor, further testing was impossible and testing was stopped.

Concrete on the top of the beam was crushed during testing, which indicated the top concrete was failing in compression. Small pieces of concrete spalled off the top of the beam, visible in Appendix B Figure 5.3.3.
Cracks were marked on the beam during testing. The east side cracks marked during testing can be found in Appendix C Figures 5.3.1(a-b). The marked west side cracks from testing are shown in Appendix C Figures 5.3.2(a-b).

5.4 – Beam 16: Post-Testing Damage Assessment

The underside of the beam experienced spalling and cracking during testing. Prior to testing, the narrowest undamaged section of the beam was 10.5”. After testing, damaged extended completely across the underside of the beam. Appendix B Figure 5.4.1 shows this spalling across the entire beam width. The view of this damage under the beam’s center is shown in Appendix B Figure 5.4.2.

Under the center of the beam, the first seven strands on the east side were visible, as seen in Appendix B Figure 5.4.3. Of these, the outer five strands were corroded. All five of these strands had broken wires from testing. The 6th and 7th strands were just barely exposed. At the exposed points, these strands showed no signs of corrosion.

The west side had outer strands two through four visible (Appendix B Figure 5.4.4). The first strand had been cut off prior to testing. All of these strands were corroded and had broken wires from testing. The second strand had broken and was hanging down.

A total of nine strands were corroded in the beam out of the fourteen strands. Five corroded strands were on the east side and four corroded strands were on the west side.
Under the east side, a stirrup became exposed during testing. Appendix B Figure 5.4.5 shows this exposed stirrup located above the damaged prestressed strands. It was firmly attached in the concrete and showed minor signs of corrosion. At the stirrup location, six strands were exposed. The first three eastern strands were corroded and bent downward out of shape from testing. The fourth and fifth strands inward from the east side were also bent downwards and were in a transition stage between corroded and uncorroded states. The sixth strand inward was barely exposed, but was not corroded in the visible section.

5.5 – Beam 16: Results

The maximum applied live loading was 38.7 kips, which corresponds to 19.4 kips at each loading point. The applied live load moment was 3,195.1 kip-in or 266.3 kip-ft.

Three full depth concrete cores were taken from beam 16. Seven partial cores were also taken, as stirrups kept being hit during coring. The average concrete core strength for the beam was 9,767 psi.

The wire potentiometers used to measure the beams deflection were removed early during testing to prevent damage to the instruments in case of a brittle failure. The wire potentiometers actually could have been left attached through the completion of the test as the failure was ductile. In future tests, the wire potentiometers were not removed. Figure 5.5.1 shows the load versus deflection graph as measured by these wire potentiometers prior to their removal. To obtain the complete load versus deflection
curve, the LVDT on the actuator was used, as shown in Figure 5.5.2. However, this LVDT measured the deflection of the cylinder, which included the deflection of the beam, the bearing pads, and the wood blocks used to transfer load to the beam. No correlation was found between the wire potentiometer data and the LVDT data to account for these additional deflections. The error between the LVDT deflection and the beam deflection was considered negligible, since the maximum deflections obtained were large enough to cause the beam to touch the floor. Therefore, the actual LVDT data were used to represent this beam’s deflection for the entire beam test. The spike in the loading near the end of the test was caused by the beam compressing against spalled concrete on the lab’s floor beneath the beam.

During the final loading cycle, the beam was nearly at the floor. With a deflection of 1’0.55”, the beam was at approximately L/32 for the testing span length. Since this is greater than L/50, it was determined the beam had failed and testing was stopped. Testing could not have continued as the beam was touching the floor. Once unloaded, beam 16 rebounded upwards to a permanent deflection of 5.5” downward from the original camber. The average pre-cracking stiffness, which was calculated to the point when cracking first occurred, was 33.3 kips/in (refer to Appendix E3.1).

The largest concrete strain measured by the west gauge was 0.002168 in/in. The largest concrete strain measured by the east gauge was 0.001806 in/in. The concrete strain gauge results are presented in Figure 5.5.4. The west strain gauge consistently measured
slightly larger strains compared to the east strain gauge. This could be attributed to the west side having less deteriorated prestressed strands.

The data provided by the clip gauges (Figure 5.5.5) were used to determine the effective prestressing force. The loading required to open the crack on the east side (green line intersections) was 17.5 kips (8.8 kips/point). The loading required to open the crack on the west side (red line intersections) was 20 kips (10 kips/point). Using this data, the average effective prestressing force was calculated to be 158.8 kips. The process used and supplemental calculations are provided in Appendix E3.1.

The strain gauge placed on the eastern deteriorated strand provided data until the strand broke during testing. Recalling this strand consisted of only two unbroken corroded wires out of the seven helically wrapped wires, failure of the strand failing was probable. This deteriorated strand failed at an applied load to the beam of 38.4 kips (19.2 kips/point). The strain associated at this loading was 0.021921 in/in. The graph of the loading versus strain for the deteriorated strand is in Figure 5.5.6.

The Response-2000 (Bentz 2000) load versus deflection curves used the concrete strength of 9,767 psi for beam 16. The dead load was removed from the Response-2000 curves so they could be compared against the live load testing results. Although the effective prestressing force was found, it was not certain how many strands were effective. It was assumed that loss of prestressing force would be similar in all beams. Beam 14 had no deteriorated strands, so the measured loss from beam 14 was assumed to
be the loss for all beams (refer to Appendix E3.2). Using the measured effective prestressing force for beam 16 and the assumed loss (from beam 14), six and a half prestressed strands were calculated to be effective (calculations are shown in Appendix E3.1). Using six and a half strands in Response-2000 gave a loading of 42.9 kips (21.5 kips/point) with a total deflection of 13.1”.

Several other Response-2000 models were run to examine the beam behavior. As strand wires broke during testing the beam became softer. Prior to softening, the best fit curve was with seven effective strands. This gave a loading of 47.6 kips (23.8 kips/point), which was never achieved. The ultimate loading was the equivalent of five and three fourths prestressed strands. This gave a loading of 37.5 kips (18.8 kips/point) with a total deflection of 12.0”. The service loading for this beam was five effective prestressed strands. Five strands gave a loading of 33.1 kips (16.6 kips/point) with a deflection of 17.1”. This deflection was not achieved since the beam was already considered failed and the testing setup did not allow that amount of deflection.

The ultimate capacity of beam 16 was found by following the provisions of the AASHTO Standard Specifications (2002). Supplemental calculations are shown in Appendix E2. The concrete strength was taken as 9,767 psi. From visual inspection, seven prestressed strands were deteriorated and the remaining strands were assumed effective. For seven effective strands, the total ultimate loading was 46.3 kips. This overestimated the maximum loading from testing by 19.7%. For the method of discounting strands adjacent to deteriorated strands there would be five effective strands, which corresponds
to a total ultimate loading of 32.7 kips. This was conservative by 15.5%. These two ultimate loadings are displayed on Figure 5.5.3 as horizontal lines.

Figure 5.5.1 – Beam 16: Load vs. Displacement before wire potentiometers detached
Figure 5.5.2 – Beam 16: Load vs. LVDT Displacement – Response-2000

Figure 5.5.3 – Beam 16: Load vs. LVDT Displacement – AASHTO
Figure 5.5.4 – Beam 16: Load vs. Concrete Strain

Figure 5.5.5 – Beam 16: Load vs. Clip Strain
Figure 5.5.6 – Beam 16: Load vs. 1st Eastern Prestressed Strand Strain at 19’6.5” from the south end
Chapter 6  
Beam 14  

6.1 – Beam 14: Pre-Testing Assessment

The second test was conducted on beam 14. Its location was the second beam inward on the western side of the bridge’s center span, as shown in Figures 3.1.2 to 3.1.4. The beam can be seen on the bridge’s plan view as shown in Figure 3.2.2. In the following description of the beam, all directions correspond to the beam’s orientation on the bridge, which is shown in Figure 4.1.1. The beam was tested in a north and south orientation. Figure 6.1 shows the west face of the beam.

![Figure 6.1 – Beam 14: West Face](image)

Beam 14 was in very good condition. Only minimal damage was found. Most of the damage was located near the beam ends and appeared to be caused during the removal process. Some cracks were present in the grouting material, likely caused from separating the beams. A longitudinal crack, shown in Appendix B Figure 6.1.1, was found running alongside the eastern prestressed strands. At this same location, a crack also extended longitudinally beneath the beam under the first prestressed strand (Appendix B Figure 6.1.2). The beam’s initial camber was measured at 2 1/8”.

45
The beam’s north end had minor damage from being removed from the bridge. A saw cut had partially extended 6” into the beam. The final saw cut placement was then readjusted 3.5” farther outward (Appendix B Figure 6.1.3). At this end, on the east side, spalled concrete at the bottom and rust from a stirrup was present (Appendix B Figure 6.1.4).

The south end of the beam had more damage than the north end caused by the beam removal. Appendix B Figure 6.1.5 shows the east side of the beam, at the south end, which had two stirrups and a prestressed strand exposed. Both stirrups and the prestressed strand showed corrosion from exposure. On the west side of this end, a corroded strand was exposed along with a corroded longitudinal rebar (Appendix B Figure 6.1.6).

Visual inspection of the beam found no reason not to assume that all fourteen strands were intact. The specified concrete strength was 5,500 psi, but based on testing done on other beams from this bridge a concrete strength of 8,000 psi was assumed (Gulistani 2010). The beam was modeled in Response-2000 (Bentz 2000) with all fourteen strands assumed effective and with complete concrete cover. The strand prestressing-strain value was taken as 5.3 micro-strain and was based on the calculated AASHTO LRFD Bridge Design Specifications prestressing loss (calculations shown in Appendix E1.1). The loading versus deflection curves from Response-2000 did not include the dead load, which allowed the results to be compared to the live loading from testing. Based on
fourteen effective strands, the total expected loading was 91.1 kips or 45.6 kips/point. The expected midspan deflection was 12.9 inches (Figure 6.2).

For the other beams tested, two theoretical loading versus deflection curves were calculated. One curve was calculated based on the subtracting the number of visible deteriorated strands from the total number of strands. The other curve assumed that any strand not visible, but next to a deteriorated strand, was also deteriorated and should also be subtracted out. For this beam, only one theoretical load versus deflection curve was calculated, since all fourteen strands were assumed effective.

![Figure 6.2 – Beam 14: Expected Load vs. Displacement](image)
6.2 – Beam 14: Testing Setup

The as-tested span length was 37’6”. This is less than the span length in the bridge, but it was shortened to the same span as the other beams to make all the tests identical. For the other beams tested, the full span length could not be used due to damage in the bearing area at the supports.

The distance between the supports and loading points was 13’9” on center as shown in Figure 4.1.2. Several other adjustments pertaining to the test setup were made between testing the first and second beams (beams 16 and 14 respectively). Two new supports were cast with dimensions of 4’ by 4’ and 1’ tall. Bearing pads were then used between the beam and the supports.

As in the previous test, concrete strain gauges were placed on the east and west ends of the top centerline. Wire potentiometers were attached to the sides of the beam at the center. No prestressed strands were available to attach a gauge, therefore no strand strain gauges were used when testing beam 14.

6.3 – Beam 14: Destructive Testing

Beam 14 was tested on June 12th, 2009. At a total loading of 50 kips (25 kips/point), cracks were first noticeable on both sides of the beam. Clip gauges were then attached across a crack on each side. The clip gauge on the west side was 14’5.5” from the north support and 1.5” from the bottom of the beam. The clip gauge on the east side was 14’1.5” from the south support and 1.75” from the bottom of the beam.
As additional cracks formed, they were marked on the beam at total loading increments of 53, 55, 60, 70, 80, and 90 kips. Appendix B Figures 6.3.1 and 6.3.2 show these marked cracks along the beam. Prestressed strand wires were heard “popping” during testing. The loading dropped slightly whenever an individual wire broke. These unexposed broken wires were believed to be from the first strand on the east side, where the initial longitudinal cracks were present. After loading the beam to 80 kips (40 kips/point), the beam was unloaded and this spalling section of concrete was chipped away in order to reveal what was occurring with this strand. By chipping away at the concrete, strand corrosion and broken wires became exposed, as shown in Appendix B Figures 6.3.3 and 6.3.4. Transverse flexural cracks formed under the beam located in the proximity of the load points (Appendix B Figure 6.3.5).

The top concrete failed from compression and started spalling off (Appendix B Figure 6.3.6). Unloading of the beam then began at a maximum total loading of 98.3 kips (49.2 kips/point), since the top concrete was popping off. The beam was then partially unloaded to a loading of 64.6 kips (32.3 kips/point). During the unloading, it was decided to reload the beam to its maximum load, allowing additional photographs to be taken. The beam was being reloaded when loud cracking and popping was heard as the beam collapsed suddenly. The collapse occurred under the northern load point at a loading of 96.3 kips (48.2 kips/point).
Cracks were marked on the beam during testing. The east side cracks marked during testing can be found in Appendix C Figures 6.3.1(a-b). The marked west side cracks from testing are shown in Appendix C Figures 6.3.2(a-b).

6.4 – Beam 14: Post-Testing Assessment

The failed section under the north loading point consisted of spalled concrete, buckled rebar, and bent prestressed strands. The failed segment of the beam along the west side is shown in Appendix B Figures 6.4.1 and 6.4.2. The damage on the east side is shown in Appendix B Figures 6.4.3 and 6.4.4. The failure damage propagated farther along the beam on the top near both sides rather than in the middle. While the spalled edge damage extended up to 3’, the narrowest section of spalled concrete on the top side in the middle was 1’2” wide.

Concrete spalled off the bottom, exposing the first five prestressed strands on the west side and the first four prestressed strands on the east side. The damage extended across the entire underside of the beam. The five western strands (Appendix B Figure 6.4.5) showed no sign of corrosion. At the collapse location, the four eastern exposed strands had small patches of corrosion in several places on the wires (Appendix B Figure 6.4.6). The corrosion on the first eastern strand, found during testing, was also in the failed region. After the beam was broken apart to be disposed of (Appendix B Figure 6.4.7), corrosion was found on the first two eastern strands.
6.5 – Beam 14: Results

The beam reached failure and collapsed suddenly at a live loading of 96.3 kips (48.2 kips/m). The east wire potentiometer displacement prior to collapse was 9.44”. The corresponding west wire potentiometer displacement was 9.33”. The load versus displacement graph is shown in Figure 6.5.1. The testing span deflection corresponds to L/43, which exceeds the L/50 failure criteria. The average pre-cracking stiffness was 44.3 kips/in, which was calculated from when cracking first occurred (refer to Appendix E3.2).

The maximum applied live load was 98.3 kips, which corresponds to 49.2 kips at each loading point. The associated maximum applied live load moment was 8,107.5 kip-in or 675.6 kip-ft. The dead load moment at midspan was 188.5 kip-in or 15.7 kip-ft; thus, the total moment applied during testing was 8,296.0 kip-in or 691.3 kip-ft. Five concrete cores were taken from the top concrete of beam 14. The average concrete core strength for the beam was 8,221 psi.

The concrete strain gauge data, graphed in Figure 6.5.3, show the west gauge reading just slightly more strain throughout the entire test. The concrete strain measured at collapse by the east strain gauge was 0.001742 in/in. The west strain gauge reading was 0.002132 in/in. The maximum strain recorded by the east and west strain gauges was 0.001801 in/in and 0.002159 in/in respectively.
The load versus clip strain gauge data is shown in Figure 6.5.4. The loading required to open the east crack (green line intersections) was 40 kips (20 kips/point). The loading required to open the west crack (red line intersections) was 38 kips (19 kips/point). The procedure used and supplemental calculations are provided in Appendix E3.2. The average effective prestress force was 344.3 kips. This corresponds to a 15% loss of prestress for beam 14.

The Response-2000 (Bentz 2000) load versus deflection curves used the concrete strength of 8,221 psi. The pre-strain value of 5.5 micro-strain was used, which was calculated from the average measured prestressing force of beam 14 (refer to Appendix E3.2). The dead load was removed from the results, allowing the results to be compared with the live load testing results. Based off initial conditions with no deteriorated strands and calculations from the average prestress force of beam 14 (see Appendix E3.2), all fourteen strands were assumed effective. Since no softening occurred during testing, this beam’s strength did not drop off with higher deflections. Fourteen strands in Response-2000 gave a maximum loading of 91.4 kips (45.7 kips/point) and a maximum deflection of 13.3”. The maximum loading using fourteen strands was similar to the ultimate loading from testing.

Deteriorated strands were still found in this beam even though it appeared in good shape. Regarding the discounting of strands adjacent to deteriorated strands, for this beam none were discounted since no deterioration was visible. However, when taking into account the beams adjacent to beam 14 on the bridge, there was a deteriorated strand visible on
the western side of beam 15 adjacent to beam 14. This corresponds to a deteriorated strand adjacent to the eastern side of beam 14, which is where the corroded strands were found in beam 14. Using the method of discounting strands adjacent to deteriorated strands, there would then be thirteen strands effective for this beam. Using Response-2000 (Bentz 2000), this corresponds to a maximum loading of 85.3 kips (42.7 kips/point) and a maximum deflection of 13.5”. This would be a more conservative approach because the beams interact on the bridge, making it is necessary to consider the adjacent beams when determining individual beam strengths. The Response-2000 load versus deflection curves are displayed on Figure 6.5.1 against the testing data.

The ultimate capacity of beam 14 was found using the provisions of the AASHTO Standard Specifications (2002). Supplemental calculations are shown in Appendix E2. The concrete strength was taken as 8,221 psi. From visual inspection, no prestressed strands were deteriorated and all were assumed to be effective. For fourteen effective strands, the total ultimate loading was 87.8 kips. This was conservative when compared to the maximum test loading by 10.7%. For the method of discounting strands adjacent to deteriorated strands with thirteen effective strands, the total ultimate loading was 82.2 kips. This was conservative by 16.4%. These two ultimate loadings are displayed on Figure 6.5.2 as horizontal lines.
Figure 6.5.1 – Beam 14: Load vs. Displacement – Response-2000

Figure 6.5.2 – Beam 14: Load vs. Displacement – AASHTO
Figure 6.5.3 – Beam 14: Load vs. Concrete Strain

Figure 6.5.4 – Beam 14: Load vs. Clip Strain
7.1 – Beam 17: Pre-Testing Assessment

The third beam to be tested was beam number 17. This beam had been located as the fifth beam inward on the western side of the bridge’s center span, as shown in Figures 3.1.2 to 3.1.4. The beam location is shown on the bridge’s plan view in Figure 3.2.2. In the following description of the beam, all directions correspond to the beam’s orientation on the bridge, which is shown in Figure 4.1.1. The beam was tested in a north and south orientation. The west face of beam 17 is shown in Figure 7.1.

![Figure 7.1 – Beam 17: West Face](image)

This beam had minor damage visible. Longitudinal cracking had occurred parallel to the outer prestressed strand. Spalled concrete had exposed small segments of prestressed strands on both sides of the beam. The initial camber was measured at 2 3/8” on the west side of the beam and 2 1/8” on the east side of the beam.

The west side of the beam had the most initial damage. Longitudinal cracking was present, as shown in Appendix B Figures 7.1.1 to 7.1.4.
Spalled concrete on the west side had exposed two corroded strands. The outer western strand was badly deteriorated and had five broken strand wires. The second strand inward was rusted and not exposed enough to distinguish the different wires composing the strand. A spalled concrete segment was barely attached next to these two western exposed strands (Appendix B Figure 7.1.5). This spalled concrete section was removed to expose any additional corroded prestressed strands and increase the accuracy of evaluating the beam’s current deterioration level. It was also hoped to expose a solid prestressed strand to attach a strain gauge. After removing the spalled area, the first four western strands were then exposed. All of these strands were badly corroded to the extent no strain gauges could be attached (Appendix B Figure 7.1.6). No further attempt was made to remove concrete to expose an intact strand as it was thought this would create artificial damage to the beam and affect the test results.

Between these longitudinal cracks and the spalled concrete, the damage extended across the majority of the beam’s western side. A rust spot had stained the west side of the beam (Appendix B Figure 7.1.7). Underneath the west side, a segment of concrete had spalled away as shown in Appendix B Figure 7.1.8.

On the east side, there was a spalled concrete section at the south end. This section had no prestressed strands visible. Near the midspan on the east side, a spalled concrete section had a corroded strand visible, as shown in Appendix B Figure 7.1.9.
The south end of the beam had minor spalling and missing concrete at the corner point, which was caused during removal of the beam. Rust stains formed on the end from the top longitudinal bars.

The north end had more damage from removal. Figure 7.1.10 shows the east side with an exposed top longitudinal bar and spalled concrete exposing the eastern prestressed strand. Two different partial saw cut lines were used to cut the north end, with the west side extending 1.75” farther outward than the rest.

Based on initial conditions, Response-2000 (Bentz 2000) was used to create the theoretical loading versus deflection curves shown in Figure 7.2. Initially, there were three corroded prestressed strands visible on beam 17. After the spalled concrete cover was removed, five corroded prestressed strands were then visible. Both of these cases were modeled in Response-2000, with eleven and nine strands assumed effective respectively.

As with the other beams, the concrete strength was taken as 8,000 psi (Gulistani 2010) and the initial prestressing-strain in the strand was set at 5.3 micro-strain. The Response-2000 loading versus deflection curves have the dead load removed, which allowed them to be compared to the live loading from testing. For eleven strands effective, the expected loading from Response-2000 (Bentz 2000) was 72.7 kips (36.4 kips/point) with a deflection of 15.3”. With the spalled concrete removed, the expected loading for the nine assumed effective strands was 60.5 kips (30.3 kips/point) and a deflection of 18.7”.
When following the method of discounting strands adjacent to deteriorated strands, as suggested by Naito (2006), two additional strands would then be discounted. One discounted strand was located adjacent to the eastern exposed strand and another adjacent to the western exposed strands. Therefore, before the spalling was removed, there would be nine effective strands modeled in Response-2000 (Bentz 2000), for an expected loading of 60.5 kips (30.3 kips/point) and a deflection of 18.7”. After the spalled concrete was removed, there would be seven effective strands following this method of discounting strands. Seven effective strands in Response-2000 corresponds to an expected loading of 47.3 kips (23.7 kips/point) with a deflection of 19.2”.

Figure 7.2 – Beam 17: Expected Load vs. Displacement
7.2 – Beam 17: Testing Setup

The tested span length was 37’6”, which was smaller than the span length on the bridge. This span length allowed all the beams to have identical span lengths with the bearing areas located at sound concrete. The distance between the supports and the load points was kept to 13’9” with 6’ between the loading points, as shown in Figure 4.1.2.

The testing setup for beam 17 was identical to the previous beams tested (beams 16 and 14). Two top concrete strain gauges and two wire potentiometers were attached along the centerline. No strand strain gauges were used, since all of the accessible strands were too badly deteriorated to attach a strain gauge.

7.3 – Beam 17: Destructive Testing

Beam 17 was tested on July 28th, 2009. Cracking was first observed at a loading of 33 kips (16.5 kips/point) on the west side and 40 kips (20 kips/point) on the east side. Clip gauges were attached across two vertical cracks on the eastern side after a loading of 43 kips (21.5 kips/point). The first clip gauge on the eastern side was located 15’8.25” from the south end and 1’3.75” from the top of the beam. The second clip gauge on the eastern side was located 20’7.5” from the south end and 1’0.75” from the top of the beam.

Cracks were marked along the beam at loading increments of 33, 40, 42, 43, 43B, 48, and 50 kips. Additional cracks were marked later during the test as 50A, 50B, and 43C since their loading was previously marked on the beam. The east side experienced vertical flexural cracking between the load points. The west side behaved unexpectedly with the
cracks mainly centralized in one location. Cracks were observed leading towards the tie rods on the west side. However, the grouting around the tie rods hid the progression of these cracks. These cracks eventually lead to the beam’s failure, which would occur at the tie rod. The east side cracks marked during testing can be found in Appendix C Figures 7.3.1(a-b). The marked west side cracks from testing are shown in Appendix C Figures 7.3.2(a-b).

Spalled concrete was pried off the east underside of the beam during testing (while the load was removed). The intention was to expose additional prestressed strands. As testing neared completion, additional concrete spalled off on the underside, exposing three additional corroded strands on the east side. The deflection of the beam during testing is shown in Appendix B Figure 7.3.1. The cracks marked along the beam prior to failure can be viewed in Appendix B Figures 7.3.2 and 7.3.3.

Top concrete along the edges failed in compression near the end of the test. At a total loading of 27.4 kips (13.7 kips/point), the beam failed between the load points and collapsed to the ground. Prior to collapse, the deflection was 1’1.92” on the west side and 1’1.36” on the east side. The deflection prior to failure had exceeded L/50; thus, the beam was already considered failed even before the collapse occurred. Upon unloading the failed beam, the remaining prestressed strands still had enough prestressing force available to raise the collapsed beam off the ground. The permanent deflection from the initial conditions, after collapse and unloading, was 11.25” on the west side and 11.5” on the east side.
7.4 – Beam 17: Post-Testing Assessment

The beam failed between the two load points at the center tie rod location on the western side (Appendix B Figure 7.4.1). The associated damage on the east side is shown in Appendix B Figure 7.4.2. After failure, all of the prestressed strands were visible on the underside. The amount of corrosion visible on the individual strands ranged from none to the entire strand being corroded.

The prestressed strand damage was symmetrical along both sides of the beam under the failure point. The first five prestressed strands on both the east and west sides of the beam were corroded. The center four prestressed strands from the beam had no corrosion visible over their exposed region. Appendix B Figures 7.4.3 and 7.4.4 show these corroded strands on the east and west sides of the beam respectively. The beam was separated into two halves at the failure location to be transported away. The cross section at failure is shown in Appendix B Figure 7.4.5. Overall, ten prestressed strands showed signs of corrosion.

7.5 – Beam 17: Results

The maximum total applied live loading beam 17 reached during testing was 50.7 kips. This corresponds to approximately 25.4 kips at each loading point. Therefore, the maximum applied live load moment was 4,181.9 kip-in or 348.5 kip-ft.
Five concrete cores were taken from the top of the beam extending down to the Styrofoam™. Two partial cores were also taken, as stirrups were hit during coring. The average concrete core strength for this beam was 8,712 psi.

The beam failed at a loading of 27.4 kips (13.7 kips/point) with a deflection of 1’1.92” on the west side and 1’1.36” on the east side. The beam’s displacement measured by the wire potentiometers is shown in Figure 7.5.1. The Response-2000 (Bentz 2000) behavior for the beam is also shown. The deflection of the beam corresponded to L/29.5 over the testing span length, which is in excess of the L/50 failure criteria. The average pre-cracking stiffness, calculated from when cracking first occurred, was 30.4 kips/in (refer to Appendix E3.3).

The concrete strain at collapse measured by the east strain gauge was 0.002274 in/in. The west strain gauge reading at collapse was 0.001605 in/in. The concrete strain gauge data is shown in Figure 7.5.3. As testing progressed, the east strain was slightly larger than the west strain throughout the remainder of the test. The maximum strain recorded by the east and west gauges were 0.002536 in/in and 0.001838 in/in respectively.

The two clip strain gauge’s data is shown in Figure 7.5.4. The loading required to open the cracks is shown by the intersecting lines where the curves slope changes. The loading required to open the southern crack (red line intersections) was 27.5 kips (13.8 kips/point). The northern crack opened (green line intersections) at 27 kips (13.5
kips/point). The average effective prestress force was calculated to be 236.7 kips. The methodology used and the supplemental calculations are provided in Appendix E3.3.

The Response-2000 (Bentz 2000) load versus deflection curves were based on the concrete strength of 8,712 psi. The prestressing-strain value of 5.5 micro-strain was used, which was calculated from the beam 14 measured prestressing force with all fourteen strands assumed effective (see Appendix E3.2). These results from Response-2000 do not include the dead load to allow comparison with the live load test results. Nine and a half prestressed strands were calculated to be effective from the obtained effective prestress force of beam 17 (see Appendix E3.3) resulting in a predicted load of 63.0 kips (31.5 kips/point) with a total deflection of 13.2”. This curve represented the best fit curve prior to softening. The beam became softer as strand wires started breaking, thereby lowering the beam’s strength. The ultimate capacity reached during testing was the equivalent of seven and a half effective strands. Seven and a half strands corresponds to a load of 49.4 kips (24.7 kips/point). The service load for this beam occurred with the equivalent of four and a half effective strands near the end of the test prior to failure. Four and a half strands corresponds to a loading of 28.8 kips (14.4 kips/point) and a deflection of 11.1”. The beam held an approximate steady loading while the deflection increased until failure occurred. The deflection at failure exceeded the deflection expected from Response-2000 for four and a half strands, but this curve still represented the beam’s capacity near the end of testing.
All but one of the initial loading versus deflection curves from Response-2000 (Bentz 2000) had overestimated the beam’s capacity. Following the discounting strands method, the initial curve with seven strands from after the spalling was removed was a good estimate of the beam’s ultimate capacity prior to softening. The maximum loading from this curve with seven strands was 47.3 kips (23.7 kips/point) while the maximum loading from testing was 50.7 kips (25.4 kips/point). The test loading had exceeded this Response-2000 curve before softening occurred, due to the beam being stiffer than the initial curve. For this beam, the method of discounting adjacent strands next to deteriorated strands produced a satisfactory estimate of the beam’s ultimate capacity. However, if this ultimate loading was sustained, the beam would fail. The safe service loading would be below the failure loading of 27.4 kips (13.7 kips/point).

The ultimate capacity of beam 17 was calculated using the provisions of the AASHTO Standard Specifications (2002). These calculations, shown in Appendix E2, used the concrete strength of 8,712 psi. There were five deteriorated strands visible, corresponding to nine assumed effective strands and a total ultimate loading of 58.8 kips. This ultimate loading exceeded the testing maximum loading by 15.9%. When following the method of discounting strands adjacent to deteriorated strands, seven strands were assumed effective. This then corresponded to a total ultimate loading of 45.9 kips, which was 9.4% conservative from the testing results. These two ultimate loadings are displayed as horizontal lines on Figure 7.5.1.
Figure 7.5.1 – Beam 17: Load vs. Displacement – Response-2000

Figure 7.5.2 – Beam 17: Load vs. Displacement – AASHTO
Figure 7.5.3 – Beam 17: Load vs. Concrete Strain

Figure 7.5.4 – Beam 17: Load vs. Clip Strain
8.1 – Beam 19: Pre-Testing Assessment

The final beam to be tested was beam 19. Its location was the seventh beam inward on the western side of the bridge’s center span, as shown in Figures 3.1.2 to 3.1.4. The beam can also be seen on the bridge’s plan view as shown in Figure 3.2.2. In the following description of the beam, all directions correspond to the beam’s orientation on the bridge, which is shown in Figure 4.1.1. The beam was tested in a north and south orientation. Figure 8.1 shows the west face of the beam.

![Figure 8.1 – Beam 19: West Face](image)

This beam had significant asymmetrical damage. The west side of the beam had major damage while the east side only had visible damage near the south end. On the west side, concrete had spalled away exposing several corroded strands, some which were broken. The initial camber of the beam was measured at 2 1/8”.
The east side damage was located at the south end of the beam. The first eastern strand was corroded and had broken, as shown in Appendix B Figure 8.1.1. A corroded second strand was barely visible. It was assumed that these strands had become exposed during beam removal process because of their exposed location at the very southern end of the beam. While the beams were in outdoor storage after being removed from the bridge, exposure to the elements had caused the prestressed strand to corrode. Therefore, these eastern strands were not considered when counting the beam’s deteriorated strands, since they would not have been visible during a bridge inspection prior to the beams removal.

Spalling on the west side of the beam extended nearly the entire length of the beam. Appendix B Figure 8.1.2 shows this spalling along the west side of the beam and exposed strands. The first three western prestressed strands were corroded (Appendix B Figure 8.1.3).

The damage under the western portion of beam 19 was the worst at the south end. Initially, the first four prestressed strands were visible and corroded extending across most of the beam, as seen in Appendix B Figure 8.1.4. The first two strands had already broken off. At the south end of the beam, seven strands were barely visible adjacent to a large delaminated section. The concrete was cracked and starting to spall off. This concrete section was removed prior to testing, thereby exposing larger sections of these first seven western prestressed strands (Appendix B Figures 8.1.5 and 8.1.6). All seven prestressed strands were badly corroded in this exposed segment. The condition of these
strands was initially unknown, and removing the concrete allowed their current state to be determined.

The western prestressed strands were both exposed and corroded as follows:

- The 1st strand was both exposed and corroded from 5’8” to 14’5.5” and from 15’9” to 34’1” from the north end. The 1st strand had broken off from 16’4’ to 30’ from the north end.
- The 2nd strand was both exposed and corroded from 6’1.5” to 11’10” and from 16’2” to 33’4” from the north end. The 2nd strand had broken off from 22’4” to 32’ from the north end.
- The 3rd strand was both exposed and corroded from 6’8” to 11’5” and from 16’6” to 32’10” from the north end.
- The 4th strand was exposed and corroded from 25’ to 32’10” from the north end.
- The 5th strand was exposed and corroded from 25’1.5” to 32’ from the north end.
- The 6th strand was exposed and corroded from 25’9” to 30’5” from the north end.
- The 7th strand was exposed and corroded from 28’3” to 29’2” from the north end.

The beam was modeled in Response-2000 (Bentz 2000) with an assumed concrete strength of 8,000 psi (Gulistani 2010) and an assumed effective prestressing-strain of 5.3 micro-strain in the strand. The Response-2000 results do not include the dead load to allow comparison with the live loading test results. There were seven corroded prestressed strands initially visible prior to testing near the south end. Therefore, this beam was modeled in Response-2000 with seven strands assumed affective and full
concrete cover for the prestressed strands. Based on the Response-2000 results, the expected loading for beam 19 with seven strands was 47.0 kips (23.5 kips/point) with a total deflection of 18.5” (Figure 8.2). Following the method of discounting strands adjacent to deteriorated strands, as suggested by Naito (2006), there would be six strands assumed effective. In Response-2000, this corresponded to an expected maximum loading of 40.2 kips (20.1 kips/point) with a total deflection of 18.8”.

![Figure 8.2 – Beam 19: Expected Load vs. Displacement](image)

This expected loading from Figure 8.2 could be considered conservative given these seven strands were corroded near the end of the beam. Throughout the remainder of the beam, three corroded strands were visible. If any of the strands became redeveloped as they neared the beam’s center, the expected loading would have been higher. However,
whether redevelopment occurred or not was unknown, so the expected loading from Response-2000 (Bentz 2000) discounted the deteriorated prestressed strands. This follows Harries’ recommendation that the contribution from redeveloped strands should be ignored (Harries 2006).

8.2 – Beam 19: Testing Setup

The testing setup for this beam was the same as the beam 14 setup. Once again, the distance between the supports and the load points was kept to 13’9” on center as shown in Figure 4.1.2. The testing span length of 37’6” was shorter than the bridge’s span length. This length allowed all the tests to be similar and have sound concrete at the bearing locations. Two top concrete strain gauges and two wire potentiometers were attached at the center of the beam. Six strand strain gauges were attached to the prestressed strands. These gauges were located as follows:

- On the 2nd western strand at 9’8.25” and at 19’11.25” from the north end.
- On the 3rd western strand at 10’3.5” and at 19’8.75” from the north end.
- On the 4th western strand at 10’2.5” and at 19’9” from the north end.

A DCDT sensor was attached to the top of the beam on the eastern side. This gauge would allow lateral movement of the beam to be recorded up to approximately 1.25” in either direction. A clip gauge was attached on the west side across an existing crack prior to testing. This clip gauge was located 11’3” from the south support and 1’1.5” down from the top of the beam.
8.3 – Beam 19: Destructive Testing

Beam 19 was tested on August 27th, 2009. Cracking was first visible after a loading of 23 kips (11.5 kips/point). The second clip gauge was attached across a crack which formed at 25 kips (12.5 kips/point). This second clip gauge was located 12’6.5” from the north support and 1’1’ from the top on the west side of the beam. However, this crack did not provide the desired data as the crack did not open up during later loadings. The clip gauge was reattached across another crack 15’4.75” from the north support and 1’0.75’ from the top on the west side of the beam. This new clip gauge location provided effective data along with the other clip gauge attached prior to testing.

Cracks were marked on the beam on the west side after total loading increments of 23, 25, 30, 33, 38, 40, 42, 46, 50, and 56 kips. The west side cracked first as this side was heavily damaged. Cracking on the east side occurred and was marked at total loading increments of 50 and 56 kips. The east side cracks marked during testing can be found in Appendix C Figures 8.3.1(a-b). The marked west side cracks from testing are shown in Appendix C Figures 8.3.2(a-b).

On the west side, a crack formed under the south load point, which would eventually extend to the top of the beam. This crack progressively opened wider throughout the remainder of the test (Appendix B Figure 8.3.1). The third western strand broke during testing and fell to the ground along the south end of the beam. Top concrete crushed under compression and could be seen popping off the top of the beam. On the east side of the beam under the southern load point, the concrete failed in compression (Appendix
The loading beam became unlevel since the low point of the failure was under the southern load point. The beam deflected westward during the test, causing the DCDT sensor to no longer be in contact with the guide beam, shown in Appendix B Figure 8.3.3. The unsymmetrical loading across the two load points combined with the lateral movement caused the test to be halted. At this point the beam was considered failed so testing stopped, avoiding the possibility of damaging the testing equipment.

8.4 – Beam 19: Post-Testing Assessment

The lateral buckling of the beam to the west was significant enough to be clearly visible without the aid of measurements. The beam deflected laterally 2.5” to the west, which can be seen when viewed along its length in Appendix B Figure 8.4.1.

The damage from the crushed concrete on the top of the beam at the east side extended from 13’5” to 16’7.5” from the south end. The damage extended on the west side of the top of the beam from 19’8.5” to 21’1.25” from the north end.

On the east underside, three prestressed strands were visible after testing (Appendix B Figure 8.4.2). These three prestressed strands showed no signs of corrosion over their exposed length.

The west underside varied in the number of exposed prestressed strands along the beam. The range was from one to seven strands exposed. Some of the strands at the south end broke and were permanently stretched from the vertical and lateral deflection during
testing. This is visible in Appendix B Figure 8.4.3. The exposed visible strand damage remained the same as before testing except under the southern load point. The bottom concrete cracked across the bottom width and several segments of concrete fell off. The first seven western strands were visible after testing under this load point. The first 4 western strands were badly deteriorated, with the first three broken off. The 5th strand inward had a corrosion patch while the 6th and 7th strands from the western side had no signs of corrosion in the exposed region from testing (Appendix B Figure 8.4.4). A corroded stirrup was visible under the south load point at 20’8.5” from the north end. Another corroded stirrup was exposed at and 22’9” from the north end (Appendix B Figure 8.4.5).

After the beam was broken apart to be disposed, corroded strands were found under the failure point at the southern load point (Appendix B Figure 8.4.6).

8.5 – Beam 19: Results

The maximum live load applied to beam 19 was 56.8 kips. The load at each load point was approximate 28.4 kips. Since the beam was failing under the southern load point, the applied load at each loading point gradually became unequal near the end of the test, but it is not possible to tell how uneven the loads became. The average maximum live load moment was 4,689.8 kip-in or 390.8 kip-ft.

Five concrete cores were taken from the beam. The average concrete core strength for beam 19 was 10,148 psi.
Beam 19 reached a maximum displacement of 9.01” on the east side and 8.84” on the west side. Figure 8.5.1 shows the displacement measured by the two wire potentiometers. The Response-2000 (Bentz 2000) results are also shown for comparison. The deflection of the testing span corresponds to L/45, which exceeds the L/50 failure criteria. The permanent deflection of the beam was 6.5” on the east side and 6.4” on the west side. The average stiffness of the beam prior to cracking was 36.1 kips/in (refer to Appendix E3.4).

This beam experienced lateral movement due to the asymmetry of the damage. The DCDT sensor only registered data up to 1.05”, when it reached its recording limit (Figure 8.5.3). The beam deflected to the west during testing a total of 2.5”. The eastern wire potentiometer displacement can be compared against the DCDT displacement, as shown in Figure 8.5.4. This graph shows an approximate linear relationship through when the DCDT sensor stopped collecting data. The lateral displacement was 13.1% of the vertical displacement. However, this relationship changes when the maximum lateral displacement (2.5”) is compared to the maximum eastern wire potentiometer displacement (9.01”). Then the lateral displacement relationship was 27.7% of the vertical displacement.

The largest concrete strain measured by the east strain gauge was 0.001636 in/in. The largest west concrete strain gauge reading was 0.000649 in/in. The concrete strain gauge data is shown in Figure 8.5.5. The east strain was always larger than the west strain during the entire test. The reason for large discrepancy between the strain gauges lies
with the lateral movement of the beam caused by the asymmetrical damage. Therefore, the beam was in biaxial bending. The vertical test loading caused both of the concrete strain gauges to be measuring compression. However, the lateral displacement caused one gauge to read additional compression while the other gage received some tension, lessening the compression readings.

The two clip strain gauge’s data can be found in Figure 8.5.6. The loading required to open the southern crack (green line intersections) on the west side was 31.5 kips (15.8 kips/point). The loading to open the northern crack (red line intersections) on the west side was 27 kips (13.5 kips/point). The process to calculate the crack openings and supplemental calculations are provided in Appendix E3.4. The average of the effective prestress force was calculated to be 208.8 kips.

Six strand strain gauges were attached prior to testing. Of these gauges only five were working when testing began. However, only four gauges provided valid data from the test. Figures 8.5.7 through 8.5.11 show the strain associated with each of the working prestressed strand strain gauges.

The Response-2000 (Bentz 2000) results used the concrete strength of 10,148 psi with a strand prestressing-strain value of 5.5 micro-strain. This micro-strain value was calculated from the measured prestress force from beam 14 with all fourteen strands assumed effective (see Appendix E3.2). These results do not include the dead load to allow comparison with the live load test results. Eight and a half prestressed strands were
calculated to be effective from the obtained effective prestress force of beam 19 (see Appendix E3.4). Eight and half strands gives a total loading of 56.5 kips (28.3 kips/point) with a total deflection of 12.6”. The best fit curve prior to softening of the beam was with nine and a half prestressed strands. Nine and a half prestressed strands gives a loading of 63.4 kips (31.7 kips/point). These loadings were never achieved, since the beam softened during testing. The ultimate loading from testing had been 56.8 kips (28.4 kips/point). This ultimate loading compares to eight and a half strands in Response-2000 for a total loading of 56.5 kips (28.3 kips/point). When testing commenced, the final service loading was the equivalent of five and a half strands in Response-2000. This corresponded to a loading of 35.8 kips (17.9 kips/point) and a deflection of 11.7”. Response-2000 overestimated the deflections seen during testing.

The ultimate capacity of beam 19 was calculated using the provisions of the AASHTO Standard Specifications (2002). The concrete strength of 8,712 psi was used. These calculations are presented in Appendix E2. There were seven deteriorated strands visible at the southern end of the beam. The remaining seven strands were assumed effective. The total ultimate loading would then be 46.4 kips, which was 18.2% conservative compared to the testing results. Following the method of discounting strands adjacent to deteriorated strands, there were six strands assumed effective. With six strands, the total ultimate loading is 39.7 kips, which is 30.2% from the testing results. These ultimate loadings are displayed as horizontal lines on Figure 8.5.2.
However, at midspan only four deteriorated strands were visible. With the remaining ten strands assumed effective, the total ultimate loading would be 66.1 kips which was 16.3% unconservative. When discounting strands adjacent to deteriorated prestressed strands, there would be nine strands assumed effective. The total ultimate loading would be 59.7 kips, which was 5% unconservative. These ultimate loadings are displayed as horizontal lines on Figure 8.5.2.

It was likely some of these deteriorated strands at the beam’s end were being redeveloped. However, when ignoring the concept of redevelopment, as suggested by Harries (2006), conservative results were obtained. Discounting adjacent strands at the end of the beam would make six strands effective, producing significantly conservative results. These results represented the service loading of the beam after strands broke, which caused softening to occur.
Figure 8.5.1 – Beam 19: Load vs. Displacement – Response-2000

Figure 8.5.2 – Beam 19: Load vs. Displacement – AASHTO
Figure 8.5.3 – Beam 19: Load vs. DCDT Displacement until its limit was reached

Figure 8.5.4 – Beam 19: DCDT Displacement vs. Eastern Wire Potentiometer Displacement until the DCDT sensor reached its limit
Figure 8.5.5 – Beam 19: Load vs. Concrete Strain

Figure 8.5.6 – Beam 19: Load vs. Clip Strain
Figure 8.5.7 – Beam 19: Load vs. 2\textsuperscript{nd} Western Prestressed Strand Strain at 9’8.25” from the north end

Figure 8.5.8 – Beam 19: Load vs. 3\textsuperscript{rd} Western Prestressed Strand Strain at 10’3.5” from the north end
Figure 8.5.9 – Beam 19: Load vs. 4<sup>th</sup> Western Prestressed Strand Strain at 10’2.5” from the north end

Figure 8.5.10 – Beam 19: Load vs. 2<sup>nd</sup> Western Prestressed Strand Strain at 19’11.25” from the north end
Figure 8.5.11 – Beam 19: Load vs. 4th Western Prestressed Strand Strain at 19’9” from the north end
Chapter 9

Estimate of Bridge Capacity from Beam Tests

9.1 – Process and Assumptions

One of the reasons for testing the beams of the LIC-310-0396 Bridge was to use the data as a means of assessing the probable, lower bound load capacity of similar bridges. As part of this study, a lower bound load capacity was calculated for the middle span of the LIC-310-0396 Bridge.

From the available pictures of the bridge’s middle span, the number of visible deteriorated strands in each beam was determined. Since these pictures did not completely document each entire beam, there could have been additional visible deteriorated strands not accounted for in the investigation.

As recommended by Naito (2006) and verified by the present study, a conservative number of strands available to carry load in each beam was determined by subtracting out the number of deteriorated strands and any strands adjacent to a deteriorated strand. The ultimate moment capacity of each individual middle span beam was determined using the provisions of the AASHTO Standard Specifications (AASHTO 2002). If the measured concrete core strength was available for a beam (Gulistani 2010), it was used to calculate the moment capacity. For beams where actual strengths were not available, 8,000 psi was used. This seemed reasonable as the lowest measured core strength was 8,221 psi. The middle span was assumed to be simply supported even though there was continuity
over the piers. Based on the bridge width of 36 feet, it was considered to be a three lane bridge for design/rating purposes. In actuality, the bridge had two lanes of traffic with shoulders on both sides.

**9.2 – Middle Span Load Ratings**

Bridges can be load rated in order to assist the Department of Transportation’s decision of whether a bridge needs load restrictions enforced or needs to be decommissioned. The load rating represents the safe live loading the bridge is able to carry and was determined by following the Load Factor Rating (LFR) method as described in The Manual for Bridge Evaluation (AASHTO 2008). This method has two different load ratings. The inventory rating identifies the loading the bridge can safely sustain for an indefinite period of time. The operating rating identifies the maximum load that can be placed on the bridge. The operating rating is used by the Department of Transportation to post legal load limits on bridges.

Each individual middle span beam was given both inventory and operating load ratings for moment capacity. Table 9.2.1 shows the Load Factor Rating for each of the middle span beams using a calculated live load distribution factor of 0.258 and a calculated impact factor of 0.308. The calculations of the Load Factor Ratings are shown in Appendix E4.

The worst beam on the middle span was beam 18, which had eight visible deteriorated prestressed strands. When discounting strands that are adjacent to deteriorated strands
for beam 18, ten strands were considered deteriorated and the remaining four strands were assumed effective. Beam 18’s inventory rating was 0.53 and its operating rating was 0.32, which is calculated in Appendix E4. Since these ratings are less than one, the strength of the beam would have been exceeded. Therefore, the likelihood of failure would have made load restrictions necessary on this bridge had it not been decommissioned.

9.3 – Middle Span Capacity
A simple analysis was conducted to determine the possibility of a progressive failure of the middle span. The lowest possible distribution factor for the middle span was 0.25, which was determined by dividing the three lanes by all twelve beams. Since the calculated distribution factor was 0.26 (Appendix E4), the difference was considered negligible and the load was assumed to be equally shared by all beams. The impact factor for moment was 0.308, as shown in Appendix E5. Using the controlling HS-44 truck, the total unfactored live load moment for the middle span was 405 kip-ft per lane. The dead load moment was 83.1 kip-ft per beam, comprised of the beam’s self weight and diaphragm weight. Table 9.3.1 shows the summarized factored and service loading results for each of the middle span beams (see Appendix E5 for calculations).

Making the simplifying assumption that the live load was equally distributed to all the beams, the resulting ultimate moment was 395.6 kip-ft per beam. This surpassed the calculated moment capacity of five middle span beams, and these five beams were considered to have failed. The ultimate moment was then recalculated with the same live
load, but now assuming it was distributed across the surviving seven middle span beams. The recalculated ultimate moment was 600.9 kip-ft per beam, which exceeded the calculated moment capacity of three additional beams. With now eight beams failed, the live load was distributed across the remaining four middle span beams. This resulted in an ultimate moment of 970.5 kip-ft per beam, which exceeded the capacity of the remaining middle span beams. Thus, a progressive failure of the entire middle span would occur under factored loads.

For the service loading analysis, the live load distribution factor was based on the live load distributed across all twelve middle span beams. The ultimate moment under service loading conditions would be 215.6 kip-ft per beam, which exceeded the capacity of beam 18. With the calculations repeated using the same live load distributed across the surviving eleven beams, an ultimate moment of 227.6 kip-ft per beam would be applied to each middle span beam. The moment capacity of all eleven remaining middle span beams exceeded this value. Therefore, no additional beams would fail and a progressive failure of the middle span would not occur under service loads.

This simple analysis showed a progressive failure might have occurred on the middle span if all the assumptions and associated load factors occurred. From the factored loading analysis, the entire middle span could progressively fail at the live loading of 405 kip-ft per lane, causing load restrictions to be necessary had this bridge not been decommissioned and replaced. From the service loading analysis, only one beam would fail in the middle span at the same live loading. Destructive testing showed the beams
had reserve capacities when following the method of discounting strands adjacent to
deteriorated strands. Therefore, some beams which were considered to have failed might
not actually fail, making this a lower bound load capacity.
### Table 9.2.1 – Summary of Middle Span Load Rating Results

<table>
<thead>
<tr>
<th>Beam Number</th>
<th># 13</th>
<th># 14</th>
<th># 15</th>
<th># 16</th>
<th># 17</th>
<th># 18</th>
<th># 19</th>
<th># 20</th>
<th># 21</th>
<th># 22</th>
<th># 23</th>
<th># 24</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Deteriorated Strands</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>West Side</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>0</td>
<td>7</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>East Side</td>
<td>0</td>
<td>0</td>
<td>4</td>
<td>4</td>
<td>1</td>
<td>8</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td><strong>Total Strands Visible</strong></td>
<td>0</td>
<td>0</td>
<td>6</td>
<td>7</td>
<td>5</td>
<td>8</td>
<td>7</td>
<td>0</td>
<td>2</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td><strong>Discounting Adjacent Strands</strong></td>
<td>0</td>
<td>1</td>
<td>8</td>
<td>9</td>
<td>7</td>
<td>10</td>
<td>8</td>
<td>0</td>
<td>3</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td><strong>Assumed Effective Strands</strong></td>
<td>14</td>
<td>13</td>
<td>6</td>
<td>5</td>
<td>7</td>
<td>4</td>
<td>6</td>
<td>14</td>
<td>11</td>
<td>13</td>
<td>14</td>
<td>14</td>
</tr>
<tr>
<td><strong>Compressive Strength, f’c (psi)</strong></td>
<td>8,000</td>
<td>8,221</td>
<td>8,519</td>
<td>9,767</td>
<td>8,712</td>
<td>9,277</td>
<td>10,148</td>
<td>8,000</td>
<td>8,000</td>
<td>8,000</td>
<td>8,000</td>
<td>8,000</td>
</tr>
<tr>
<td><strong>Moment Capacity (k-ft)</strong></td>
<td>626.4</td>
<td>590.7</td>
<td>295.3</td>
<td>250.5</td>
<td>341.5</td>
<td>201.9</td>
<td>298.4</td>
<td>626.4</td>
<td>509.3</td>
<td>588.3</td>
<td>626.4</td>
<td>626.4</td>
</tr>
<tr>
<td><strong>Operating Load Rating</strong></td>
<td>2.91</td>
<td>2.71</td>
<td>1.05</td>
<td>0.80</td>
<td>1.31</td>
<td>0.53</td>
<td>1.07</td>
<td>2.91</td>
<td>2.26</td>
<td>2.70</td>
<td>2.91</td>
<td>2.91</td>
</tr>
<tr>
<td><strong>Inventory Load Rating</strong></td>
<td>1.75</td>
<td>1.63</td>
<td>0.63</td>
<td>0.48</td>
<td>0.79</td>
<td>0.32</td>
<td>0.64</td>
<td>1.75</td>
<td>1.35</td>
<td>1.62</td>
<td>1.75</td>
<td>1.75</td>
</tr>
</tbody>
</table>
### Table 9.3.1 – Summary of Middle Span Capacity Results

<table>
<thead>
<tr>
<th>Beam Number</th>
<th># 13</th>
<th># 14</th>
<th># 15</th>
<th># 16</th>
<th># 17</th>
<th># 18</th>
<th># 19</th>
<th># 20</th>
<th># 21</th>
<th># 22</th>
<th># 23</th>
<th># 24</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deteriorated Strands</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>West Side</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>0</td>
<td>7</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>East Side</td>
<td>0</td>
<td>0</td>
<td>4</td>
<td>4</td>
<td>1</td>
<td>8</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Total Strands Visible</td>
<td>0</td>
<td>0</td>
<td>6</td>
<td>7</td>
<td>5</td>
<td>8</td>
<td>7</td>
<td>0</td>
<td>2</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Total Deteriorated Strands Discounting Adjacent Strands</td>
<td>0</td>
<td>1</td>
<td>8</td>
<td>9</td>
<td>7</td>
<td>10</td>
<td>8</td>
<td>0</td>
<td>3</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Assumed Effective Strands</td>
<td>14</td>
<td>13</td>
<td>6</td>
<td>5</td>
<td>7</td>
<td>4</td>
<td>6</td>
<td>14</td>
<td>11</td>
<td>13</td>
<td>14</td>
<td>14</td>
</tr>
<tr>
<td>Compressive Strength, f'c (psi)</td>
<td>8,000</td>
<td>8,221</td>
<td>8,519</td>
<td>9,767</td>
<td>8,712</td>
<td>9,277</td>
<td>10,148</td>
<td>8,000</td>
<td>8,000</td>
<td>8,000</td>
<td>8,000</td>
<td>8,000</td>
</tr>
<tr>
<td>Moment Capacity (k-ft)</td>
<td>626.4</td>
<td>590.7</td>
<td>295.3</td>
<td>250.5</td>
<td>341.5</td>
<td>201.9</td>
<td>298.4</td>
<td>626.4</td>
<td>509.3</td>
<td>588.3</td>
<td>626.4</td>
<td>626.4</td>
</tr>
<tr>
<td>Factored Loading Analysis</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moment = 395.6 k-ft / beam</td>
<td>Ok</td>
<td>Ok</td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
<td>Ok</td>
<td>Ok</td>
<td>Ok</td>
<td>Ok</td>
<td>Ok</td>
</tr>
<tr>
<td>Moment = 600.9 k-ft / beam</td>
<td>Ok</td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
<td>Ok</td>
<td>Failed</td>
<td>Failed</td>
<td>Ok</td>
<td>Ok</td>
</tr>
<tr>
<td>Moment = 970.5 k-ft / beam</td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
</tr>
<tr>
<td>Service Loading Analysis</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moment = 215.6 k-ft / beam</td>
<td>Ok</td>
<td>Ok</td>
<td>Ok</td>
<td>Ok</td>
<td>Ok</td>
<td>Failed</td>
<td>Ok</td>
<td>Ok</td>
<td>Ok</td>
<td>Ok</td>
<td>Ok</td>
<td>Ok</td>
</tr>
<tr>
<td>Moment = 227.6 k-ft / beam</td>
<td>Ok</td>
<td>Ok</td>
<td>Ok</td>
<td>Ok</td>
<td>Ok</td>
<td>Failed</td>
<td>Ok</td>
<td>Ok</td>
<td>Ok</td>
<td>Ok</td>
<td>Ok</td>
<td>Ok</td>
</tr>
</tbody>
</table>
Chapter 10

Results Summary

10.1 – Results Summary

This project tested four beams removed from a deteriorated bridge. Table 10.1.1 summarizes the number of corroded strands found in each beam from pre-testing and post-testing. The pre-testing strand results came from a visual inspection of the beam with any loose concrete removed. After testing the beams, it was typical to find additional corroded prestressed strands which became visible due to spalling of the concrete during testing. The destructive testing results are shown in Table 10.1.2. After destructive testing, samples were taken of prestressed strands to be tested by Ohio University. The results obtained by Gulistani (2010) are shown in Table 10.1.3.

The wire potentiometer load versus deflection for each of the beams is shown in Figure 10.1.1. The load versus deflection for the LVDT was used for beam 16 (see Chapter 5), since the wire potentiometers were taken off early during the test because of fears of a brittle failure.

While each of the beam tests ended under different circumstances (beams 14 and 17 collapsed, beam 16 deflected to the floor, and beam 19 had lateral instability), each beam was considered to have failed by the definition of the beam’s deflection exceeding one fiftieth of the span length. All of the beams with visible damage (beams 16, 17, 19) experienced softening during testing, which is also visible in Figure 10.1.1.
The maximum concrete strain from each of the beam tests is shown in Figure 10.1.2. The concrete strains for each of the beams were similar and ranged between 1,636 and 2,536 micro-strain.

The pre-testing and post-testing Response-2000 (Bentz 2000) results are presented in Tables 10.1.4 and 10.1.5 respectively. Response-2000 was used to develop load versus deflection curves based on the number of assumed effective strands. The number of effective strands was determined by a visible inspection and by discounting strands adjacent to deteriorated prestressed strands, as suggested by Naito (2006). After the beams were tested, the loading versus deflection curves from Response-2000 were altered by changing the number of strands, the concrete strength, and the prestressing-strain, based on data from the destructive test. The generated curves then would more accurately represent the behavior of the beam.

The provisions of the AASHTO Standard Specifications (AASHTO 2002) were used to calculate the ultimate capacity for each beam, discounting corroded strands and strands adjacent to corroded strands. This was found to provide conservative estimates of beam strength. The summarized results are shown in Table 10.1.6 while the calculations are shown in Appendix E2. However, for beams 16, 17, and 19 the ultimate loading was reached early during testing. The softening of these beams caused larger deflection to be reached at smaller loadings. The safe service loading for the beams was less than this ultimate loading which would then make the results unconservative for these three beams.
The prestress loss for the beams was calculated using three different methods. These methods were the AASHTO LRFD Bridge Design Specifications (AASHTO 2007), the AASHTO Standard Specifications (AASHTO 2002), and the PCI Design Handbook (PCI 2004). The calculations to find the prestress loss are shown in Appendix E1 while the summarized results are shown in Table 10.1.7. The AASHTO Standard Specifications and the PCI Design Handbook methods yielded almost identical results of 25.4% and 49.6% respectively. The AASHTO LRFD Bridge Design Specifications gave a smaller prestress loss of 18.5%. This method produced a more accurate prestress loss than the other methods. The beam 14 prestress loss with all fourteen prestressed strands assumed effective was 14.95%. Therefore, all of the methods used to solve for the prestress loss were on the conservative side.

![Graph](image)

**Figure 10.1.1** – Load vs. Displacement Beam Comparison
Figure 10.1.2 – Load vs. Concrete Strain Comparison
Table 10.1.1 – Summary of Deteriorated Prestressed Strands

<table>
<thead>
<tr>
<th></th>
<th>Beam 16</th>
<th>Beam 14</th>
<th>Beam 17</th>
<th>Beam 19</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>East</td>
<td>West</td>
<td>East</td>
<td>West</td>
</tr>
<tr>
<td>Pre-Testing Visible Corroded Strands</td>
<td>4</td>
<td>3</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>After Spalling Removed</td>
<td>4</td>
<td>3</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Pre-Testing Visible Corroded Strands</td>
<td>7</td>
<td>0</td>
<td>5</td>
<td>7</td>
</tr>
<tr>
<td>Total Pre-Testing Visible Corroded Strands</td>
<td>9</td>
<td>1</td>
<td>7</td>
<td>8</td>
</tr>
<tr>
<td>Discounting Adjacent Strands Method</td>
<td>5</td>
<td>4</td>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td>Total Post-Testing Corroded Strands</td>
<td>9</td>
<td>2</td>
<td>10</td>
<td>8</td>
</tr>
</tbody>
</table>
### Table 10.1.2 – Summary of Destructive Testing Results

<table>
<thead>
<tr>
<th></th>
<th>Beam 16</th>
<th></th>
<th>Beam 14</th>
<th></th>
<th>Beam 17</th>
<th></th>
<th>Beam 19</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>East</td>
<td>West</td>
<td>East</td>
<td>West</td>
<td>East</td>
<td>West</td>
<td>East</td>
<td>West</td>
</tr>
<tr>
<td>Loading Cracks Observed (kips)</td>
<td>32</td>
<td>32</td>
<td>50</td>
<td>50</td>
<td>40</td>
<td>33</td>
<td>50</td>
<td>23</td>
</tr>
<tr>
<td>Loading Cracks Observed (kips/point)</td>
<td>16</td>
<td>16</td>
<td>25</td>
<td>25</td>
<td>20</td>
<td>16.5</td>
<td>25</td>
<td>11.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure Loading (kips)</td>
<td></td>
<td>96.3</td>
<td></td>
<td>27.4</td>
<td></td>
<td>n/a</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure Loading (kips/point)</td>
<td></td>
<td>48.2</td>
<td></td>
<td>13.7</td>
<td></td>
<td>n/a</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum Total Loading (kips)</td>
<td>38.7</td>
<td>98.3</td>
<td></td>
<td>50.7</td>
<td></td>
<td>56.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum Total Loading (kips/point)</td>
<td>19.4</td>
<td>49.2</td>
<td></td>
<td>25.4</td>
<td></td>
<td>28.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum Applied Live Moment (k-ft)</td>
<td>266.3</td>
<td>675.6</td>
<td></td>
<td>348.5</td>
<td></td>
<td>390.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lateral Displacement (in)</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
<td>2.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Displacement at Failure (in)</td>
<td>n/a</td>
<td>9.44</td>
<td>9.33</td>
<td>13.92</td>
<td>13.36</td>
<td>n/a</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum Displacement (in)</td>
<td>12.55</td>
<td>9.44</td>
<td>9.33</td>
<td>13.92</td>
<td>13.36</td>
<td>9.01</td>
<td>8.84</td>
<td></td>
</tr>
<tr>
<td>Deflection Criteria L/?</td>
<td>L/32</td>
<td>L/43</td>
<td>L/29.5</td>
<td></td>
<td>L/45</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete Strain at Failure (in/in)</td>
<td>n/a</td>
<td>0.001742</td>
<td>0.002132</td>
<td>0.002274</td>
<td>0.001605</td>
<td>n/a</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum Concrete Strain (in/in)</td>
<td>0.001806</td>
<td>0.002168</td>
<td>0.001801</td>
<td>0.002159</td>
<td>0.002536</td>
<td>0.001838</td>
<td>0.001636</td>
<td>0.000649</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Effective Prestress Force (kips)</td>
<td>158.8</td>
<td>344.3</td>
<td></td>
<td>236.7</td>
<td></td>
<td>208.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Core Compressive Strength, f′c (psi)</td>
<td>9,767</td>
<td>8,221</td>
<td></td>
<td>8,712</td>
<td></td>
<td>10,148</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Modulus of Elasticity, E (ksi)</td>
<td>5,633</td>
<td>5,168</td>
<td></td>
<td>5,320</td>
<td></td>
<td>5,742</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moment of Inertia, I (in⁴)</td>
<td>11,597</td>
<td>13,824</td>
<td></td>
<td>10,007</td>
<td></td>
<td>10,644</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pre-Cracking Stiffness (kips/in)</td>
<td>33.3</td>
<td>44.3</td>
<td></td>
<td>30.4</td>
<td></td>
<td>36.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strand Specimen Number</td>
<td>Sample Length</td>
<td>Gage Length</td>
<td>Visual Strand Classification</td>
<td>Young’s Modulus E (ksi)</td>
<td>Yield Strength Fy (ksi)</td>
<td>Ultimate Strength Fu (ksi)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>------------------------</td>
<td>---------------</td>
<td>-------------</td>
<td>-----------------------------</td>
<td>-------------------------</td>
<td>------------------------</td>
<td>---------------------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam 16</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sample 1</td>
<td>20”</td>
<td>14.5”</td>
<td>Class 3</td>
<td>21,010.0</td>
<td>Not achieved</td>
<td>154.953</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sample 2</td>
<td>22”</td>
<td>16”</td>
<td>Class 2</td>
<td>28,173.0</td>
<td>250.0</td>
<td>264.984</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sample 3</td>
<td>19.5”</td>
<td>15”</td>
<td>Class 2</td>
<td>29,234.0</td>
<td>265.0</td>
<td>278.380</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam 17</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sample 1</td>
<td>25”</td>
<td>20”</td>
<td>Class 1</td>
<td>28,804.0</td>
<td>265.0</td>
<td>291.033</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sample 2</td>
<td>23.5”</td>
<td>19”</td>
<td>Class 1</td>
<td>30,676.0</td>
<td>260.0</td>
<td>287.561</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sample 3</td>
<td>18”</td>
<td>14”</td>
<td>Class 2</td>
<td>26,390.0</td>
<td>250.0</td>
<td>264.093</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sample 4</td>
<td>17.5”</td>
<td>13”</td>
<td>Class 3</td>
<td>31,950.0</td>
<td>255.0</td>
<td>266.115</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sample 5</td>
<td>24.5”</td>
<td>21.5”</td>
<td>Class 1</td>
<td>32,377.0</td>
<td>267.0</td>
<td>290.850</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sample 6</td>
<td>15”</td>
<td>10.5”</td>
<td>Class 2</td>
<td>33,010.0</td>
<td>263.0</td>
<td>283.344</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sample 7</td>
<td>21”</td>
<td>18.5”</td>
<td>Class 2</td>
<td>31,450.0</td>
<td>265.0</td>
<td>291.422</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sample 8</td>
<td>26”</td>
<td>20”</td>
<td>Class 3</td>
<td>30,352.0</td>
<td>Not achieved</td>
<td>235.897</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam 19</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sample 1</td>
<td>15.3”</td>
<td>12”</td>
<td>Class 3</td>
<td>27,062.0</td>
<td>Not achieved</td>
<td>217.096</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sample 2</td>
<td>22.5”</td>
<td>17.5”</td>
<td>Class 3</td>
<td>27,002.0</td>
<td>Not achieved</td>
<td>177.635</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 10.1.4 – Summary of Pre-Testing Response-2000 Behavioral Results

<table>
<thead>
<tr>
<th></th>
<th>Beam 16</th>
<th></th>
<th>Beam 14</th>
<th></th>
<th>Beam 17</th>
<th></th>
<th>Beam 19</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>East</td>
<td>West</td>
<td>East</td>
<td>West</td>
<td>East</td>
<td>West</td>
<td>East</td>
<td>West</td>
</tr>
<tr>
<td>Initially Assumed Intact Strands</td>
<td>7</td>
<td>14</td>
<td>11</td>
<td>7</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Expected Maximum Loading (kips)</td>
<td>47.0</td>
<td>91.1</td>
<td>72.7</td>
<td>47.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Expected Maximum Displacement (in)</td>
<td>18.3</td>
<td>12.9</td>
<td>15.3</td>
<td>18.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Discounting Adjacent Strands Method</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Initially Assumed Intact Strands</td>
<td>5</td>
<td>13</td>
<td>9</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Expected Maximum Loading (kips)</td>
<td>33.1</td>
<td>85.0</td>
<td>60.5</td>
<td>40.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Expected Maximum Displacement (in)</td>
<td>18.5</td>
<td>13.9</td>
<td>18.7</td>
<td>18.8</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>After Spalling Removed</td>
<td>n/a</td>
<td>n/a</td>
<td>9</td>
<td>7</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Initially Assumed Intact Strands</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Expected Maximum Loading (kips)</td>
<td></td>
<td></td>
<td>60.5</td>
<td>47.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Expected Maximum Displacement (in)</td>
<td></td>
<td></td>
<td>18.7</td>
<td>18.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Discounting Adjacent Strands Method</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>After Spalling Removed</td>
<td>n/a</td>
<td>n/a</td>
<td>7</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Initially Assumed Intact Strands</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Expected Maximum Loading (kips)</td>
<td></td>
<td></td>
<td>47.3</td>
<td>40.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Expected Maximum Displacement (in)</td>
<td></td>
<td></td>
<td>19.2</td>
<td>18.8</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Table 10.1.5 – Summary of Post-Testing Response-2000 Behavioral Results

<table>
<thead>
<tr>
<th></th>
<th>Beam 16</th>
<th>Beam 14</th>
<th>Beam 17</th>
<th>Beam 19</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>East Side</td>
<td>West Side</td>
<td>East Side</td>
<td>West Side</td>
</tr>
<tr>
<td>Calculated Strands Before Softening Using AASHTO Prestress Loss</td>
<td>6.75</td>
<td>14</td>
<td>10</td>
<td>9</td>
</tr>
<tr>
<td>Maximum Loading (kips)</td>
<td>44.6</td>
<td>91.3</td>
<td>67.3</td>
<td>61.4</td>
</tr>
<tr>
<td>Maximum Displacement (in)</td>
<td>13.2</td>
<td>13.6</td>
<td>18.4</td>
<td>18.3</td>
</tr>
<tr>
<td>Calculated Strands Before Softening Using Beam 14 Prestress Loss</td>
<td>6.5</td>
<td>14</td>
<td>9.5</td>
<td>8.5</td>
</tr>
<tr>
<td>Maximum Loading (kips)</td>
<td>42.9</td>
<td>91.4</td>
<td>63.0</td>
<td>56.5</td>
</tr>
<tr>
<td>Maximum Displacement (in)</td>
<td>13.1</td>
<td>13.3</td>
<td>13.2</td>
<td>12.6</td>
</tr>
<tr>
<td>Strands for Best Fit Before Softening Using Beam 14 Prestress Loss</td>
<td>7</td>
<td>14</td>
<td>9.5</td>
<td>9</td>
</tr>
<tr>
<td>Maximum Loading (kips)</td>
<td>47.6</td>
<td>91.4</td>
<td>63.0</td>
<td>63.4</td>
</tr>
<tr>
<td>Maximum Displacement (in)</td>
<td>19.1</td>
<td>13.3</td>
<td>13.2</td>
<td>13.5</td>
</tr>
<tr>
<td>Strands for Ultimate Capacity Using Beam 14 Prestress Loss</td>
<td>5.75</td>
<td>14</td>
<td>7.5</td>
<td>8.5</td>
</tr>
<tr>
<td>Maximum Loading (kips)</td>
<td>37.5</td>
<td>91.4</td>
<td>49.4</td>
<td>56.5</td>
</tr>
<tr>
<td>Maximum Displacement (in)</td>
<td>12.0</td>
<td>13.3</td>
<td>12.3</td>
<td>12.6</td>
</tr>
<tr>
<td>Strands for Near End of Testing Using Beam 14 Prestress Loss</td>
<td>5</td>
<td>14</td>
<td>4.5</td>
<td>5.5</td>
</tr>
<tr>
<td>Maximum Loading (kips)</td>
<td>33.1</td>
<td>91.4</td>
<td>28.8</td>
<td>35.8</td>
</tr>
<tr>
<td>Maximum Displacement (in)</td>
<td>17.1</td>
<td>13.3</td>
<td>11.1</td>
<td>11.7</td>
</tr>
<tr>
<td></td>
<td>Beam 16</td>
<td>Beam 14</td>
<td>Beam 17</td>
<td>Beam 19</td>
</tr>
<tr>
<td>-----------------------</td>
<td>---------</td>
<td>---------</td>
<td>---------</td>
<td>---------</td>
</tr>
<tr>
<td><strong>Strands Initially Assumed Effective by</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Discounting Visible Strands</strong></td>
<td>7</td>
<td>14</td>
<td>11</td>
<td>7</td>
</tr>
<tr>
<td><strong>Strands Initially Assumed Effective by</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Discounting Adjacent Strands</strong></td>
<td>5</td>
<td>13</td>
<td>7</td>
<td>6</td>
</tr>
<tr>
<td><strong>Testing Ultimate Loading (kips)</strong></td>
<td>38.7</td>
<td>98.3</td>
<td>50.7</td>
<td>56.8</td>
</tr>
<tr>
<td><strong>AASHTO Standard Specifications</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Strands Effective for Ultimate Capacity by Discounting Visible Strands</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Ultimate Loading (kips)</strong></td>
<td>46.3</td>
<td>87.8</td>
<td>58.8</td>
<td>46.4</td>
</tr>
<tr>
<td><strong>% Conservative</strong></td>
<td>19.7 %</td>
<td>10.7 %</td>
<td>15.9 %</td>
<td>18.2 %</td>
</tr>
<tr>
<td><strong>Unconservative</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Conservative</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>AASHTO Standard Specifications</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Strands Effective for Ultimate Capacity by Discounting Adjacent Strands</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Ultimate Loading (kips)</strong></td>
<td>32.7</td>
<td>82.2</td>
<td>45.9</td>
<td>39.7</td>
</tr>
<tr>
<td><strong>% Conservative</strong></td>
<td>15.5 %</td>
<td>16.4 %</td>
<td>9.4 %</td>
<td>30.2 %</td>
</tr>
<tr>
<td><strong>Conservative</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Response-2000</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Strands Effective for Ultimate Capacity</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Ultimate Loading (kips)</strong></td>
<td>37.5</td>
<td>91.4</td>
<td>49.4</td>
<td>56.5</td>
</tr>
<tr>
<td><strong>% Conservative</strong></td>
<td>3.1 %</td>
<td>7.0 %</td>
<td>2.5 %</td>
<td>0.7 %</td>
</tr>
<tr>
<td><strong>Conservative</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 10.1.7 – Summary of Prestress Loss Results

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Loss (ksi)</td>
<td>28.25</td>
<td>34.85</td>
<td>47.88</td>
<td>49.47</td>
</tr>
<tr>
<td>Effective Prestress (ksi)</td>
<td>160.75</td>
<td>154.15</td>
<td>141.12</td>
<td>139.53</td>
</tr>
<tr>
<td>% Loss</td>
<td>14.95</td>
<td>18.44</td>
<td>25.33</td>
<td>26.18</td>
</tr>
</tbody>
</table>
Chapter 11
Summary, Conclusions, and Recommendations

11.1 – Summary

Four deteriorated adjacent prestressed concrete box beams were removed from the decommissioned LIC-310-0396 Bridge in Licking County, Ohio. There was a wide range of deterioration present among the beams. Some beams had little or no deterioration while other beams had major deterioration. The deterioration ranged from longitudinal cracking to spalled concrete with exposed, corroded prestressed strands.

The box beams had in service span lengths of 37 feet 6 inches and were 36 inches wide by 17 inches deep. Initially, the beams contained fourteen prestressed strands, which were ½” diameter, stress relieved strands. The prestressed strands were arranged in one layer, with the stirrups located above the strands. This stirrup arrangement was a standard design practice when the bridge was designed. However, now this design detail is no longer used and has been changed to placing the stirrups below the prestressed strands. By placing the stirrups below the prestressed strands, it not only provides additional concrete cover to the strands, but it also helps resist the bottom concrete cover from cracking and spalling.

Beam 14 was the benchmark for all of the tested beams, since it had no visible deteriorated strands or spalled concrete. Only longitudinal cracking was visible on this beam. The beam had the least amount of damage and achieved the highest loading of all
the tested beams. This beam failed at a loading which exceeded that of the AASHTO LRFD Bridge Design Specifications (AASHTO 2007) and the AASHTO Standard Specifications (AASHTO 2002). Although the beam was designed under Standard Specifications, the capacity was calculated using both AASHTO specifications to be thorough. The beam also failed at a load exceeding that predicted by Response-2000 (Bentz 2000). After destructive testing, two deteriorated prestressed strands were found. Even with these deteriorated strands the beam still exceeded the ultimate loadings. The prestress loss was calculated by following the provisions of the AASHTO LRFD Bridge Design Specifications, the AASHTO Standard Specifications, and the PCI Design Handbook. All three methods were conservative compared to the calculated prestress loss from destructively testing beam 14. The provisions of the AASHTO LRFD Bridge Design Specifications produced close results, while the other two methods were much more conservative.

The other three beams destructively tested had significantly smaller ultimate loadings and experienced softening effects. Their prestress force was smaller and they deflected farther during testing. The deflections obtained during destructive testing for each beam had exceeded the L/50 deflection failure criteria, signifying the beams were very ductile.

The AASHTO Standard Specifications ultimate capacity was conservative for all four destructively tested beams when using the method of discounting strands adjacent to deteriorated strands, as suggested by Naito (2006). The amount of conservativeness varied from 9.4% to 30.2%, which was attributed to some of the discounted deteriorated
prestressed strands still being able to provide strength. However, due to softening, if this load was sustained on the deteriorated beams they would eventually fail.

A simple analysis was conducted using both factored and service loadings on the middle span of the bridge. The ultimate moment for each middle span beam from the LIC-310-0396 Bridge was obtained. The number of deteriorated strands for each beam was based on available pictures. Then the method of discounting adjacent strands was applied, as suggested by Naito (2006). Continuity across the piers was ignored. The dead load was applied per beam. The designed live load of 405 kip-ft per lane was distributed across all of the surviving beams. Under service loading, one beam from the middle span would fail. A progressive failure of the entire middle span would occur under factored loads. The associated load factors and assumptions would all have to occur in order to have a progressive failure. Since beams would fail in either load case, load restrictions would have been necessary had the bridge not been decommissioned.

Load ratings can be applied to each of the middle span beams. Beam 18 was the worst beam on the middle span. When discounting adjacent strands, four strands would be assumed affective. Following the Load Factor Rating (LFR) method, the operating rating would be 0.53 while the inventory rating would be 0.32. As this method determines the maximum live loading for the bridge, these low ratings signify that load restrictions would have been necessary for the bridge had it not been decommissioned.
11.2 – Conclusions

Beams without any visible deterioration can still have deterioration present within the beam. Beam 14 had no visible deteriorated strands prior to testing, but two deteriorated strands were found after testing. The beam still exceeded the AASHTO Standard Specifications ultimate loading without accounting for these two unknown deteriorated strands.

The AASHTO LRFD Bridge Design Specifications determines the prestress loss to be 18.44%, which exceeded the testing prestress loss of 14.95%. The AASHTO Standard Specifications method prestress loss was 25.33% while the PCI Handbook method prestress loss was 26.18%. With prestress loss varying on the testing conditions, such as humidity, the AASHTO LRFD Bridge Design Specifications is the best method to determine the prestress loss. Testing results performed by Tadros determined the prestress loss for prestressed high strength concrete would be more accurate with the method adapted by the AASHTO LRFD Bridge Design Specifications (Tadros 2003).

Each of the beams deflections had reached or exceeded L/45. The damaged beams had significantly lower capacities and had experienced softening during testing. All of the beams were ductile. The failure mode for each failed beam was in flexure.

The method of discounting strands adjacent to deteriorated strands is reasonable, as suggested by Naito (2006). When present, spalled concrete was removed to better determine the number of deteriorated strands. Any deteriorated prestressed strands found
were assumed to be ineffective along the entire span length. The adjacent strands next to any deteriorated prestressed strands were also assumed ineffective and discounted along the entire span length. The remaining prestressed strands in the beam were then assumed to be intact and effective. Redevelopment of the deteriorated strands was ignored, as was suggested by Harries (2006). The capacity of the beam was then determined by following the provisions of the AASHTO Standard Specifications, the computer program Response-2000 (Bentz 2000), or other appropriate engineering methods. By discounting strands adjacent to deteriorated strands, the ultimate loading from the AASHTO Standard Specifications was achieved during each beam’s destructive test with conservative results. From the University of Pittsburg destructive tests, Harries recommended an additional strands loss of 25% to account for deteriorated strands that are not visible (Harries 2006).

11.3 – Recommendations

The Ohio Department of Transportation, along with other state departments of transportation, needs to accurately determine the strength of their adjacent prestressed concrete box beam bridges. When following the provisions of the AASHTO Standard Specifications to determine the ultimate loading, the number of effective prestressed strands should be determined by using the method of discounting prestressed strands adjacent to deteriorated prestressed strands. Any deteriorated strands and the strands adjacent to it should be discounted and deemed ineffective along the entire span length. Note that this method should also be used across the shear keys. If an edge strand is deteriorated, both the adjacent strand in the same beam and the adjacent edge strand in
the next beam should be discounted. Spalled concrete should be removed to better
determine the number of deteriorated prestressed strands. During destructive testing, this
method of determining effective prestressed strands provided a conservative ultimate
loading.

Deteriorated prestressed strands can still provide strength, since the individual strand
wires in tension would still provide strength to the beam. It is uncertain the amount of
capacity that a deteriorated strands can provide the beam. Since a strand’s condition is
uncertain along the entire span length, determining the capacity of these strands is
impossible. Theoretically, the strand could redevelop when the damage occurs near the
beam ends. However, relying on this redevelopment in determining the beam’s capacity
would bring uncertainty into question and would not be conservative. Therefore,
discounting a prestressed strand entirely along the span length when deterioration is
present is conservative, which was also recommended by Harries (Harries 2006).

The bridge was decommissioned, which appears to have been a correct decision. Had the
beams been loaded to the service loading of the bridge, a middle span beam would have
failed. However, if the factored loading case were to occur on the bridge, the entire
middle span may have failed. For the factored load case, once the most deteriorated
beam failed, the surviving beams would have been unable to withstand the increased
demand. Therefore, a progressive failure may have occurred. With load restrictions
being enforced, the bridge might not have been decommissioned. However, there is no
way to ensure that an accidental overloading would never occur and cause the middle span to fail.

11.4 – Future Research Recommendations

Future studies of adjacent prestressed concrete box beams should address situations that were not able to be tested during this study. Future studies of box beams would provide valuable additional information.

The beams tested in this research project were shallow and contained only one row of prestressed strands. It would be advisable for future studies to incorporate box beams with different box beam geometries, span lengths, and strand arrangements. Deeper box beams with strands in a second layer should also be investigated further. The second layer strands have a larger bottom cover, but are closer to the box void section. It is possible for the void sections to fill with water and initiate strand corrosion from within the beam.

Testing conducted on an actual bridge would be beneficial. This would allow the interaction of beams and the distribution of loads among the adjacent damaged and undamaged beams to be investigated. Lateral movements of tested beams would be restrained by their adjacent beams, but would then place an applied lateral load on these adjacent beams. This situation presented on a bridge would be hard to mimic in a testing frame.
The number of effective strands in a box beam in service is an unknown factor in determining the beam’s strength. Future research should continue to investigate better ways to determine the number of effective strands remaining. Attaching strain gauges to the prestressed strands during beam fabrication would allow the stress in each strand to be determined. However, attaching strain gauges to all the strands and recording/analyzing the data would be an overwhelming task, which would not be an economical solution to the issue. However the number of effective prestressed strands is determined, the solution not only needs to be accurate but economical.

Damaged box beam bridges in service can be rehabilitated instead of decommissioned depending on the severity of the damage sustained. Knowing when to repair or replace a deteriorated bridge depends on knowledge learned from previous test results. However, over time deterioration will still continue and will then lower the capacity.
References

American Association of State Highway and Transportation Officials.

American Association of State Highway and Transportation Officials.

Washington, D.C.: American Association of State Highway and Transportation
Officials.

Modified Compression Field Theory*. Version 1.0.5.

Gulistani, Aziz A. March 2010. Forensic Investigation of Prestressed Concrete Box
Beams from LIC-310 Bridge. A thesis presented to the faculty of the Russ
College of Engineering and Technology of Ohio University in partial fulfillment
of the requirements for the degree Master of Science.

Harries, K. A. 2009. Structural Testing of Prestressed Concrete Girders from the Lake

on De-commissioned Girders from the Lake View Drive Bridge*. The Pennsylvania
Department of Transportation, Dept. of Civil and Environmental Engineering,
Univ. of Pittsburgh, Pittsburgh, Rep. No. CE/ST 33.


Appendix A

Bridge Drawings
GENERAL NOTES

1. General: The plans, elevations, sections, and working drawings constituting this specification shall be read in conjunction with the specification.

2. Special Instructions: All work not specifically mentioned herein shall be executed in accordance with the latest edition of the American Standards Association Specifications for Highway Bridges.

3. Specifications: The specifications, plans, and section drawings are a part of this contract and shall be read in conjunction with the general specifications.

4. Changes: Any changes or modifications to the plans or specifications shall be made in writing and shall be signed by the architect.

5. Execution: The work shall be completed within 90 days from the date of award.

6. Payment: Payment shall be made in accordance with the provisions of the contract.

7. Inspection: The work shall be inspected periodically by the architect.

8. Completion: The work shall be completed in a workmanlike manner and in accordance with the plans and specifications.

9. Records: The contractor shall keep complete records of all work done and shall furnish the architect with copies of all such records.

10. Materials: The contractor shall furnish all materials and labor necessary to complete the work.

11. Subcontracts: The contractor shall not subcontract any portion of the work without the written consent of the architect.

12. Liens: The contractor shall give a written notice of lien to the owner if the contractor is not paid within 30 days of the date of the notice.

13. Indemnification: The contractor shall indemnify and hold harmless the owner from any claims or demands arising out of the work.

14. Final Payment: Final payment shall be made upon the submission of a statement by the contractor certifying that all work has been completed.

15. Termination: The contract may be terminated by the owner upon 30 days notice for cause.


GENERAL SUMMARY

[Table and chart details]

[Figure and diagram details]
### CALCULATIONS

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Quantity</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>First Item</td>
<td>100</td>
<td></td>
<td>100</td>
</tr>
<tr>
<td>1.2</td>
<td>Second Item</td>
<td>200</td>
<td></td>
<td>200</td>
</tr>
<tr>
<td>1.3</td>
<td>Third Item</td>
<td>300</td>
<td></td>
<td>300</td>
</tr>
</tbody>
</table>

### OTHER CALCULATIONS

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>First Example</td>
<td>123</td>
</tr>
<tr>
<td>2.2</td>
<td>Second Example</td>
<td>456</td>
</tr>
<tr>
<td>2.3</td>
<td>Third Example</td>
<td>789</td>
</tr>
</tbody>
</table>
Appendix B

Beam Photographs

Appendix B1 – Beam 16 Photographs
Appendix B2 – Beam 14 Photographs
Appendix B3 – Beam 17 Photographs
Appendix B4 – Beam 19 Photographs
Appendix B1

Beam 16 Photographs
### Beam 16: Pre-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.1.1</td>
<td>West underside 3’6” to 20’10” from north end</td>
<td>First western prestressed strand broken and hanging loose. The strand was cut off prior to testing.</td>
<td><img src="image1.png" alt="Photograph" /></td>
</tr>
<tr>
<td>5.1.2</td>
<td>West underside 3’6” to 20’10” from north end</td>
<td>First western prestressed strand broken and hanging loose. The strand was cut off prior to testing.</td>
<td><img src="image2.png" alt="Photograph" /></td>
</tr>
<tr>
<td>5.1.3</td>
<td>East side 0’ to 4’9” from south end</td>
<td>Spalled concrete and corroded strand hanging loose to the end of the beam.</td>
<td><img src="image3.png" alt="Photograph" /></td>
</tr>
<tr>
<td>Figure</td>
<td>Location</td>
<td>Description</td>
<td>Photograph</td>
</tr>
<tr>
<td>--------</td>
<td>----------</td>
<td>-------------</td>
<td>------------</td>
</tr>
<tr>
<td>5.1.4</td>
<td>East side 1’ to 2’6” from north end</td>
<td>Spalled concrete exposing two corroded strands and two corroded stirrups.</td>
<td><img src="image1.jpg" alt="Photograph" /></td>
</tr>
<tr>
<td>5.1.5</td>
<td>East underside 6’ to 8’ from north end</td>
<td>Two exposed corroded strands.</td>
<td><img src="image2.jpg" alt="Photograph" /></td>
</tr>
<tr>
<td>5.1.6</td>
<td>East underside 10’6” to 13’6” from north end</td>
<td>Three exposed corroded strands.</td>
<td><img src="image3.jpg" alt="Photograph" /></td>
</tr>
<tr>
<td>Figure</td>
<td>Location</td>
<td>Description</td>
<td>Photograph</td>
</tr>
<tr>
<td>--------</td>
<td>----------</td>
<td>---------------------------------</td>
<td>------------</td>
</tr>
<tr>
<td>5.1.7</td>
<td>East underside 16’ to 19’ from north end</td>
<td>Four exposed corroded strands.</td>
<td></td>
</tr>
<tr>
<td>5.1.8</td>
<td>East underside 6’8” to 10’ from south end</td>
<td>Three exposed corroded strands.</td>
<td></td>
</tr>
<tr>
<td>5.1.9</td>
<td>East underside 3’6” to south end</td>
<td>Three exposed corroded strands.</td>
<td></td>
</tr>
</tbody>
</table>
### Beam 16: Pre-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location Description</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.1.10</td>
<td>West underside 15’6” to 17’ from north end</td>
<td>Two exposed corroded strands (the first strand was cut off).</td>
<td><img src="image1" alt="Photograph" /></td>
</tr>
<tr>
<td>5.1.11</td>
<td>West underside 17’6” from north end</td>
<td>Three exposed corroded strands (the first strand was cut off).</td>
<td><img src="image2" alt="Photograph" /></td>
</tr>
<tr>
<td>5.1.12</td>
<td>West underside 8’10” to 10’ from north end</td>
<td>Three exposed corroded strands (the first strand was cut off).</td>
<td><img src="image3" alt="Photograph" /></td>
</tr>
</tbody>
</table>
### Beam 16: Pre-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.1.13</td>
<td>West underside 7’ from north end</td>
<td>Two exposed corroded strands (the first strand was cut off).</td>
<td><img src="image1" alt="Photograph" /></td>
</tr>
<tr>
<td>5.1.14</td>
<td>West underside 5’6” to north end</td>
<td>Location where the first strand was cut off.</td>
<td><img src="image2" alt="Photograph" /></td>
</tr>
<tr>
<td>5.1.15</td>
<td>Underside 10’ to 11’ from north end</td>
<td>Narrow undamaged width of 13.5”.</td>
<td><img src="image3" alt="Photograph" /></td>
</tr>
</tbody>
</table>
## Beam 16: Pre-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.1.16</td>
<td>Underside 16’ to 17’ from north end</td>
<td>Narrowest undamaged width of 10.5”.</td>
<td><img src="image" alt="Photograph" /></td>
</tr>
<tr>
<td>5.1.17</td>
<td>East side 7’3” to 12’ from south end</td>
<td>Saw cut into beam 2” at the bottom of the east side.</td>
<td><img src="image" alt="Photograph" /></td>
</tr>
</tbody>
</table>
### Beam 16: Testing Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.3.1</td>
<td>West side</td>
<td>Beam’s deflection during final loading cycle.</td>
<td></td>
</tr>
<tr>
<td>5.3.2</td>
<td>West side</td>
<td>Beam’s deflection during final loading cycle.</td>
<td></td>
</tr>
<tr>
<td>5.3.3</td>
<td>Top center</td>
<td>Compression failure of top concrete.</td>
<td></td>
</tr>
</tbody>
</table>
### Beam 16: Post-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.4.1</td>
<td>Underside center from east side</td>
<td>Spalled concrete across entire beam width. Eastern 5\textsuperscript{th}, 6\textsuperscript{th}, and 7\textsuperscript{th} strands are shown exposed. Western 2\textsuperscript{nd} strand is broken and hanging loose.</td>
<td><img src="image1" alt="Photograph" /></td>
</tr>
<tr>
<td>5.4.2</td>
<td>Underside center</td>
<td>Damage view looking south.</td>
<td><img src="image2" alt="Photograph" /></td>
</tr>
<tr>
<td>5.4.3</td>
<td>East underside 16’ to 21’ from north end</td>
<td>Eastern 5 strands corroded.</td>
<td><img src="image3" alt="Photograph" /></td>
</tr>
</tbody>
</table>
### Beam 16: Post-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.4.4</td>
<td>West underside</td>
<td>Western 4 strands corroded (the first strand was hanging down and cut off).</td>
<td></td>
</tr>
<tr>
<td>5.4.5</td>
<td>East underside</td>
<td>Exposed stirrup above deteriorated strands.</td>
<td></td>
</tr>
</tbody>
</table>
Appendix B2

Beam 14 Photographs
# Beam 14: Pre-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.1.1</td>
<td>East side 19’6” to 21’0” from south end</td>
<td>Longitudinal crack beside first eastern strand.</td>
<td><img src="image1.png" alt="Photograph" /></td>
</tr>
<tr>
<td>6.1.2</td>
<td>Underside 19’6” to 21’0” from south end</td>
<td>Longitudinal crack under first eastern strand.</td>
<td><img src="image2.png" alt="Photograph" /></td>
</tr>
<tr>
<td>6.1.3</td>
<td>North end at west corner</td>
<td>Saw cut 6” into beam before being readjusted 3.5” outward.</td>
<td><img src="image3.png" alt="Photograph" /></td>
</tr>
</tbody>
</table>
### Beam 14: Pre-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.1.4</td>
<td>East side 34'3” to north end</td>
<td>11” section of spalled concrete. Rust from corroded stirrup at 36’3”</td>
</tr>
<tr>
<td>6.1.5</td>
<td>East side 0’ to 2’8” from south end</td>
<td>Exposed corroded stirrups and strand.</td>
</tr>
<tr>
<td>6.1.6</td>
<td>West side 35’ to south end</td>
<td>Corroded strand exposed for 1’5”. Corroded rebar exposed for 6”.</td>
</tr>
</tbody>
</table>
# Beam 14: Testing Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.3.1</td>
<td>West side 11’5” to 21’ from north end</td>
<td>Cracks marked along the west side.</td>
<td><img src="image" alt="Photograph" /></td>
</tr>
<tr>
<td>6.3.2</td>
<td>East side through 25’6” from south end</td>
<td>Cracks marked along the east side.</td>
<td><img src="image" alt="Photograph" /></td>
</tr>
<tr>
<td>6.3.3</td>
<td>East side 19’5” to 20’6” from south end</td>
<td>Corroded strand exposed concrete was chipped off.</td>
<td><img src="image" alt="Photograph" /></td>
</tr>
</tbody>
</table>
## Beam 14: Testing Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.3.4</td>
<td>East side 20'3” from south end</td>
<td>Corroded strand with broken wires.</td>
<td><img src="image1.png" alt="Image" /></td>
</tr>
<tr>
<td>6.3.5</td>
<td>Underside view from 20’ from south end</td>
<td>Transverse cracking under beam.</td>
<td><img src="image2.png" alt="Image" /></td>
</tr>
<tr>
<td>6.3.6</td>
<td>Top of beam between load points</td>
<td>Top concrete failing in compression.</td>
<td><img src="image3.png" alt="Image" /></td>
</tr>
</tbody>
</table>
### Beam 14: Post-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.4.1</td>
<td>West side 12’ to 16’ from north end</td>
<td>West side damage from testing. Top damage was from 13’ to 16’. Bottom damage was from 10’4” to 14’2”.</td>
<td><img src="image" alt="Photograph" /></td>
</tr>
<tr>
<td>6.4.2</td>
<td>West side 12’ to 16’ from north end</td>
<td>West side damage after spalled concrete cleared away.</td>
<td><img src="image" alt="Photograph" /></td>
</tr>
<tr>
<td>6.4.3</td>
<td>East side 19’ to 23’6” from south end</td>
<td>East side damage from testing. Top damage was from 19’8” to 22’7”. Bottom damage was from 18’6” to 24’0”.</td>
<td><img src="image" alt="Photograph" /></td>
</tr>
</tbody>
</table>
# Beam 14: Post-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.4.4</td>
<td>East side 19’ to 23’6” from south end</td>
<td>East side damage after spalled concrete cleared away.</td>
<td><img src="image1" alt="East side damage" /></td>
</tr>
<tr>
<td>6.4.5</td>
<td>West underside</td>
<td>Five western strands exposed without corrosion.</td>
<td><img src="image2" alt="Five western strands" /></td>
</tr>
<tr>
<td>6.4.6</td>
<td>East underside</td>
<td>Four eastern strands exposed with small patches of corrosion.</td>
<td><img src="image3" alt="Four eastern strands" /></td>
</tr>
</tbody>
</table>
### Beam 14: Post-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.4.7</td>
<td>Failure point from testing</td>
<td>Cross section of the beam after being separated.</td>
<td></td>
</tr>
</tbody>
</table>
Appendix B3

Beam 17 Photographs
### Beam 17: Pre-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.1.1</td>
<td>West side 7’4” to 10’7.75” from the north end</td>
<td>Longitudinal cracking.</td>
<td><img src="image1.png" alt="Photograph" /></td>
</tr>
<tr>
<td>7.1.2</td>
<td>West side from 10’8.5” to 13’1” from the north end</td>
<td>Longitudinal cracking.</td>
<td><img src="image2.png" alt="Photograph" /></td>
</tr>
<tr>
<td>7.1.3</td>
<td>West side 23’11” to 26’9.5” from the north end</td>
<td>Longitudinal cracking.</td>
<td><img src="image3.png" alt="Photograph" /></td>
</tr>
<tr>
<td>7.1.4</td>
<td>West side 27’4” to 29’3.5” from the north end</td>
<td>Longitudinal cracking.</td>
<td><img src="image4.png" alt="Photograph" /></td>
</tr>
</tbody>
</table>
### Beam 17: Pre-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.1.5</td>
<td>West underside</td>
<td>Spalled from 15’9” to 20’11”. First strand visible from 16’7” to 18’3” and 19’7” to 20’10”. Second strand visible from 19’9” to 20’1”</td>
<td><img src="image1" alt="Photograph" /></td>
</tr>
<tr>
<td>7.1.6</td>
<td>West underside</td>
<td>Four exposed corroded strands after spalling was removed.</td>
<td><img src="image2" alt="Photograph" /></td>
</tr>
</tbody>
</table>
### Beam 17: Pre-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.1.7</td>
<td>West side 7’ from north end</td>
<td>Rust stain on west beam face.</td>
</tr>
<tr>
<td>7.1.8</td>
<td>West underside from 11’ to 13’ from north end</td>
<td>Spalling 6” to 9” inward from the beam edge.</td>
</tr>
<tr>
<td>7.1.9</td>
<td>East side 19’5.5” to 21’5” from the south end</td>
<td>Spalled concrete and 1st eastern corroded strand exposed for 1’1”.</td>
</tr>
</tbody>
</table>
### Beam 17: Pre-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.1.10</td>
<td>East side 35’ from the south end to the north end of the beam</td>
<td>Top longitudinal bar exposed at 35’10” to the end. Prestressed strand exposed at 35’2” to the end</td>
<td><img src="image.jpg" alt="Photograph" /></td>
</tr>
</tbody>
</table>
# Beam 17: Testing Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.3.1</td>
<td>West side view</td>
<td>Beam deflection during testing. Measuring tape shows the initial height of the beam.</td>
<td><img src="Image1" alt="Photograph" /></td>
</tr>
<tr>
<td>7.3.2</td>
<td>West side 15’ to 19’ from the north end</td>
<td>West side cracking prior to failure.</td>
<td><img src="Image2" alt="Photograph" /></td>
</tr>
<tr>
<td>7.3.3</td>
<td>East Side 16’ to 20’ from south end</td>
<td>East side cracking prior to failure.</td>
<td><img src="Image3" alt="Photograph" /></td>
</tr>
</tbody>
</table>
# Beam 17: Post-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.4.1</td>
<td>West side damage 18’ to 19’6” from the north end</td>
<td>Cracking passing through the tie rod.</td>
<td><img src="image1.png" alt="Photograph" /></td>
</tr>
<tr>
<td>7.4.2</td>
<td>East side from 16’ to 18’ from the south end</td>
<td>East side damage from testing at the tie rod.</td>
<td><img src="image2.png" alt="Photograph" /></td>
</tr>
<tr>
<td>7.4.3</td>
<td>East underside</td>
<td>Five eastern strands corroded.</td>
<td><img src="image3.png" alt="Photograph" /></td>
</tr>
</tbody>
</table>
Beam 17: Post-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.4.4</td>
<td>West underside</td>
<td>Five western strands corroded.</td>
<td></td>
</tr>
<tr>
<td>7.4.5</td>
<td>Failure point along tie rod from testing</td>
<td>Cross section of the beam after being separated.</td>
<td></td>
</tr>
</tbody>
</table>
Appendix B4

Beam 19 Photographs
## Beam 19: Pre-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.1.1</td>
<td>East side from 0’ to 5’7” from the south end</td>
<td>First strands exposed corroded from 1’2.5” to 4’0” and broken at 2’2.5”. Second strand exposed from 1’11” to 2’3”.</td>
<td><img src="image1" alt="Photograph" /></td>
</tr>
<tr>
<td>8.1.2</td>
<td>West side from 26’ to the north end</td>
<td>Spalled concrete and exposed strands. Spalling began at 4’8” from the north end.</td>
<td><img src="image2" alt="Photograph" /></td>
</tr>
<tr>
<td>8.1.3</td>
<td>West underside from 5’8” to 12’ from the north end</td>
<td>Three exposed corroded strands.</td>
<td><img src="image3" alt="Photograph" /></td>
</tr>
</tbody>
</table>
Beam 19: Pre-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.1.4</td>
<td>West Underside view 25’ from north end looking south</td>
<td>Four corroded strands exposed (the first two were broken off).</td>
<td><img src="image1" alt="Photograph" /></td>
</tr>
<tr>
<td>8.1.5</td>
<td>West Underside view 25’ from north end looking south</td>
<td>After delaminated concrete was broken off, seven corroded strands were exposed (the first two were broken off).</td>
<td><img src="image2" alt="Photograph" /></td>
</tr>
<tr>
<td>8.1.6</td>
<td>West Underside looking north</td>
<td>After delaminated concrete was broken off, seven corroded strands were exposed (the first two were broken off).</td>
<td><img src="image3" alt="Photograph" /></td>
</tr>
</tbody>
</table>
### Beam 19: Testing Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.3.1</td>
<td>West side from 19’ to 21’ from north end</td>
<td>Major crack opening under the southern load point which extended to the top of the beam.</td>
<td><img src="image1.png" alt="Photograph" /></td>
</tr>
<tr>
<td>8.3.2</td>
<td>East side from 13’ to 16’ from south end</td>
<td>East side damage under southern loading point.</td>
<td><img src="image2.png" alt="Photograph" /></td>
</tr>
<tr>
<td>8.3.3</td>
<td>East side at 16’1.75” from south end</td>
<td>East side damage and DCDT sensor no longer connected to the guide beam.</td>
<td><img src="image3.png" alt="Photograph" /></td>
</tr>
</tbody>
</table>
### Beam 19: Post-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.4.1</td>
<td>View along beam looking north</td>
<td>Lateral buckling of the beam to the west.</td>
<td><img src="image" alt="Photograph" /></td>
</tr>
<tr>
<td>8.4.2</td>
<td>East side from 13’7” to 14’4” from the south end</td>
<td>Three exposed strands without deterioration.</td>
<td><img src="image" alt="Photograph" /></td>
</tr>
</tbody>
</table>
### Beam 19: Post-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.4.3</td>
<td>West side 23’ from the north end and looking south</td>
<td>Western seven corroded strands.</td>
<td><img src="image" alt="Photograph" /></td>
</tr>
<tr>
<td>8.4.4</td>
<td>West side 24’ from the north end and looking north</td>
<td>Western four corroded strands and the next three good strands. Corroded stirrup at 22’9” from the north end.</td>
<td><img src="image" alt="Photograph" /></td>
</tr>
<tr>
<td>8.4.5</td>
<td>West side 22’9” from north end</td>
<td>Deteriorated stirrup above deteriorated strands.</td>
<td><img src="image" alt="Photograph" /></td>
</tr>
</tbody>
</table>
# Beam 19: Post-Testing Damage Photographs

<table>
<thead>
<tr>
<th>Figure</th>
<th>Location</th>
<th>Description</th>
<th>Photograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.4.6</td>
<td>Southern loading point</td>
<td>Cross section of the beam after being separated.</td>
<td></td>
</tr>
</tbody>
</table>
Appendix C

Beam Crack Diagrams

Appendix C1 – Beam 16 Crack Diagrams
Appendix C2 – Beam 14 Crack Diagrams
Appendix C3 – Beam 17 Crack Diagrams
Appendix C4 – Beam 19 Crack Diagrams
Appendix C1

Beam 16 Crack Diagrams
Appendix C Figure 5.3.1(b) – Beam 16 East Side Cracks
Appendix C Figure 5.3.2(a) – Beam 16 West Side Cracks
Appendix C2

Beam 14 Crack Diagrams
Appendix C Figure 6.3.1(a) – Beam 14 East Side Cracks
Appendix C Figure 6.3.1(b) – Beam 14 East Side Cracks
Appendix C Figure 6.3.2(a) – Beam 14 West Side Cracks
Appendix C Figure 6.3.2(b) – Beam 14 West Side Cracks
Appendix C3

Beam 17 Crack Diagrams
Appendix C Figure 7.3.2(a) – Beam 17 West Side Cracks
Appendix C Figure 7.3.2(b) – Beam 17 West Side Cracks
Appendix C4

Beam 19 Crack Diagrams
Appendix C Figure 8.3.1(a) – Beam 19 East Side Cracks
Appendix C Figure 8.3.1(b) – Beam 19 East Side Cracks
Appendix C Figure 8.3.2(a) – Beam 19 West Side Cracks
Appendix D

Response-2000 Sample Input

The following data was used in Response-2000 (Bentz 2000) in determining the loading versus deflection curves.

**Concrete Cross Section I-Beam**

Top flange width = 36 inches  
Height of I-beam = 17 inches  
Web thickness = 10 inches  
Top flange thickness = 5 inches  
Bottom flange thickness = 5 inches  
Bottom flange width = 36 inches

**Concrete Details**

Cylinder Strength = 8,000 psi (Assumed value. Actual value varied per beam)  
Tension Strength = Auto 325 psi  
Peak Strain = 3.00 ms  
Aggregate Size = 0.75 in  
Tension Stiff Factor = 1.0  
Base Curve = Popovics/Thorenfeldt/Collins  
Comp. Softening = Vecchio-Collins 1986  
Tension Stiffening = Bentz 1999
**Prestressing Steel Details**

Ramberg-Osgood A = 0.030  
Ramberg-Osgood B = 121.0  
Ramberg-Osgood C = 6.0  
Elastic Modulus = 29008 ksi  
Ultimate Strength = 270 ksi  
Rupture Strain = 43  
Predefined Type = 270 ksi Stress Relieved

**Tendons**

Number of Strands = 14 (Number varied)  
Strand Designation = S.5 (Area = 0.153 in²)  
Pre-strain = 5.5 ms (from beam 14 prestress or 5.3 ms from AASHTO prestress loss)  
Distance from Bottom = 1.8 in  
Slope of Tendon = 0.0  
Tendon Type = Stress Relieved

**Transverse Reinforcement**

Stirrup Spacing = 24 in  
Bar Designation = #4  
Distance to Top = 15.2 inches  
Distance to Bottom = 2.3 inches  
Bar Type = Open Stirrup
Rebar Type = Long

**Full Member Properties**

Length subjected to Shear = 165 inches

Constant Moment zone on right = 36 inches

Constant Shear Analysis (Point Loads)

Left Side Properties = Support on bottom

Right Side Properties = Load on continuous beam, load on top
Appendix E

Calculations

Appendix E1 – Loss of Prestress

Appendix E2 – Ultimate Capacity

Appendix E3 – Effective Prestress Force
Appendix E1

Prestress Loss

The calculated prestress loss for the prestressed beams is presented in the following sections using three different methods. The different methods used to find the prestress loss include the AASHTO LRFD Bridge Design Specifications (AASHTO 2007), the AASHTO Standard Specifications (AASHTO 2002), and the PCI Design Handbook (PCI 2004).

Appendix E1.1 – AASHTO LRFD Bridge Design Specifications Prestress Loss

Appendix E1.2 – AASHTO Standard Specifications Prestress Loss

Appendix E1.3 – PCI Design Handbook Prestress Loss
Appendix E1.1

AASHTO LRFD Bridge Design Specifications Prestress Loss

The prestress loss calculated by using the provisions of the AASHTO LRFD Bridge Design Specifications is presented over the following pages (AASHTO 2007).

Elastic Shortening Alternative Equation

The elastic shortening loss for pretensioned members was calculated using the alternative equation C5.9.5.2.3a-1 of the AASHTO LRFD Bridge Design Specifications (AASHTO 2007).

\[
\Delta f_{pES} = \frac{A_{ps} f_{pbt} (I_g + e_m^2 A_g) - e_m M_g A_g}{A_{ps} (I_g + e_m^2 A_g) + \frac{A_g I_g E_{ci}}{E_p}}
\]

Where:

\[
A_{ps} = 14 \text{ strands} \times 0.153 \frac{in^2}{\text{strand}} = 2.142 \text{ in}^2
\]

\[
A_g = 430 \text{ in}^2
\]

\[
E_{ci} = \frac{(150 \text{ pcf})^{1.5} \times 33 \times \sqrt{4,000 \text{ ksi}}}{1000 \frac{lb}{k}} = 3,834 \text{ ksi}
\]

\[
E_p = 28,900 \text{ ksi}
\]

\[
e_m = \frac{17 \text{ in}}{2} - 1.5 \text{ in} - \frac{0.5 \text{ in}}{2} = 6.75 \text{ in}
\]

\[
f_{pbt} = 0.75 f_{pu} = 0.70 \times 270 \text{ ksi} = 189.0 \text{ ksi}
\]

\[
I_g = 13,995.8 \text{ in}^4
\]
Approximate Estimate of Time-Dependent Losses

The long-term prestress loss can be approximated using equation 5.9.5.3-1 of the AASHTO LRFD Bridge Design Specifications. This formula includes the losses due to concrete creep, concrete shrinkage, and steel relaxation (AASHTO 2007).

\[
\Delta f_{PLT} = 10.0 \frac{f_{pl} A_{ps}}{A_g} \gamma_h \gamma_{st} + 12.0 \gamma_h \gamma_{st} + \Delta f_{PR}
\]

Where:
\[ f_{pi} = 0.70f_{pu} = 0.70 \times 270 \text{ ksi} = 189.0 \text{ ksi} \]

\[ A_{ps} = 14 \times 0.153 \text{ in}^2 = 2.142 \text{ in}^2 \]

\[ A_g = 430 \text{ in}^2 \]

\[ \gamma_h = 1.7 - 0.01H = 1.7 - 0.01 \times 70 = 1 \]

\[ \gamma_{st} = \frac{5}{1 + f'_{cl}} = \frac{5}{1 + 8} = 0.56 \]

\[ \Delta f_{pR} = 10 \text{ ksi} \]

\[ \Delta f_{pLT} = 10.0 \times \frac{189.0 \text{ ksi} \times 2.142 \text{ in}^2}{430 \text{ in}^2} \times 1 \times 0.56 + 12.0 \times 1 \times 0.56 + 10 \text{ ksi} \]

\[ \Delta f_{pLT} = 21.90 \text{ ksi} \]

**Total Loss of Prestress**

The total prestress loss for pretensioned members is calculated using equation 5.9.5.1-1 of the AASHTO LRFD Bridge Design Specifications (AASHTO 2007)

\[ \Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} \]

\[ \Delta f_{pT} = 12.95 \text{ ksi} + 21.90 \text{ ksi} \]

\[ \Delta f_{pT} = 35.85 \text{ ksi} \]

**Effective Prestress**

\[ = 0.70 \times 270 \text{ ksi} - 34.85 \text{ ksi} = 154.15 \text{ ksi} \]

\[ \text{Effective Prestress} = \frac{154.15 \text{ ksi}}{270 \text{ ksi}} = 0.57f_{pu} \]

\[ \text{Prestress Loss} = \frac{\Delta f_{pT}}{0.70f_{pu}} = \frac{34.85 \text{ ksi}}{0.70 \times 270 \text{ ksi}} \times 100 = 18.44 \% \]

\[ \text{Strand Prestrain} = \frac{154.15 \text{ ksi}}{28,900 \text{ ksi}} = 0.00533 \]
Appendix E1.2
AASHTO Standard Specifications Prestress Loss

The prestress loss calculated using the provisions of the AASHTO Standard Specifications is presented over the following pages (AASHTO 2002).

Shrinkage

The shrinkage loss can be found using equation 9-4 of the AASHTO Standard Specifications (AASHTO 2002).

\[ SH = 17,000 - 150RH \]

Where:

\[ RH = 70\% \]

\[ SH = (17,000 - 150 \times 70) \times \frac{k}{1,000 \text{ lb}} = 6.50 \text{ ksi} \]

Elastic Shortening

The elastic shortening loss can be found using equation 9-6 of the AASHTO Standard Specifications (AASHTO 2002).

\[ ES = \frac{E_s}{E_{cl}} f_{c_{ir}} \]

Where:

\[ E_s = 28,900 \text{ ksi} \]

\[ E_{cl} = \frac{(150 \text{ pcf})^{1.5} \times 33 \times \sqrt{4,000 \text{ ksi}}}{1000 \frac{\text{lb}}{k}} = 3,834 \text{ ksi} \]
\[ P_{si} = 14 \text{ strands} \times 0.153 \frac{in^2}{strand} \times 0.63 \times 270 \text{ ksi} = 364.35 \text{ ksi} \]

\[ A = 430 \text{ in}^2 \]

\[ I = 13,995.8 \text{ in}^4 \]

\[ e_c = \frac{17 \text{ in}}{2} - 1.5 \text{ in} - \frac{0.5 \text{ in}}{2} = 6.75 \text{ in} \]

Diaphragm wt. = \((18 \text{ in} + 12 \text{ in}) \times (36 \text{ in} - 10 \text{ in}) \times (17 \text{ in} - 10\text{ in}) \times \frac{0.150 \text{ kcf}}{12^3 \frac{in^3}{ft^3}}\]

Diaphragm wt. = 0.474 \frac{k}{\text{diaphragm}}

\[ M_{diaphragm} = \frac{PL}{4} = \frac{0.474 \frac{k}{\text{diaphragm}} \times \left(2 \times 165 \text{ in} + 6 \text{ ft} \times \frac{12 \text{ in}}{ft}\right)}{4} \]

\[ M_{diaphragm} = 47.63 \text{ k-in} \]

\[ M_{beam} = \frac{wL^2}{8} = \frac{0.150 \frac{kcf}{12^3 \frac{in^3}{ft^3}} \times 430 \text{ in}^2 \times \left(2 \times 165 \text{ in} + 6 \text{ ft} \times \frac{12 \text{ in}}{ft}\right)^2}{8} \]

\[ M_{beam} = 754.01 \text{ k-in} \]

\[ M_g = M_{diaphragm} + M_{beam} = 47.63 \text{ k-in} + 754.01 \text{ k-in} = 801.64 \text{ k-in} \]

\[ f_{cir} = \frac{P_{si}}{A} + \frac{P_{si}e_c^2}{I} - \frac{(M_g + M_d)e_c}{I} \]

\[ f_{cir} = \frac{364.35 \text{ ksi}}{430 \text{ in}^2} + \frac{364.35 \text{ ksi} \times (6.75 \text{ in})^2}{13,995.8 \text{ in}^4} - \frac{(801.64 \text{ k-in} + 26.17 \text{ k-in}) \times 6.75 \text{ in}}{13,995.8 \text{ in}^4} \]

\[ f_{cir} = 1.647 \text{ ksi} \]

\[ ES = \frac{28,900 \text{ ksi}}{3,834 \text{ ksi}} \times 1.647 \text{ ksi} = 12.41 \text{ ksi} \]
Creep

The creep loss is found using equation 9-9 of the AASHTO Standard Specifications (AASHTO 2002).

\[ CR_c = 12f_{cr} - 7f_{cd} \]

Where:

\[ f_{cr} = 1.647 \text{ ksi} \]

\[ \text{Barrier wt.} = 2 \times \left( 90 \frac{\text{lbs}}{\text{ft}} \times \frac{k}{1,000 \text{ lb}} \times \frac{ft}{12 \text{ in}} \right) \times \frac{1}{12 \text{ beams}} = 0.00125 \frac{k}{\text{in}} \]

\[ M_{\text{barrier}} = \frac{wL^2}{8} = \frac{0.00125 \frac{k}{\text{in}} \times \left( 2 \times 165 \text{ in} + 6 \text{ ft} \times \frac{12 \text{ in}}{\text{ft}} \right)^2}{8} = 25.25 \text{ k-in} \]

\[ \text{Wearing surface wt.} = 3 \text{ in} \times \frac{0.150 \text{ kcf}}{12^3 \frac{\text{in}^3}{\text{ft}^3}} \times 36 \text{ ft} \times \frac{12 \text{ in}}{\text{ft}} \times \frac{1}{12 \text{ beams}} \]

\[ \text{Wearing surface wt.} = 0.009375 \frac{k}{\text{in}} \]

\[ M_{\text{wearing surface}} = \frac{wL^2}{8} = \frac{0.009375 \frac{k}{\text{in}} \times \left( 2 \times 165 \text{ in} + 6 \text{ ft} \times \frac{12 \text{ in}}{\text{ft}} \right)^2}{8} \]

\[ M_{\text{wearing surface}} = 189.38 \text{ k-in} \]

\[ M_{\text{SDL}} = M_{\text{barrier}} + M_{\text{wearing surface}} = 25.25 \text{ k-in} + 189.38 \text{ k-in} = 214.63 \text{ k-in} \]

\[ f_{cds} = \frac{M_{\text{SDL}}e_c}{I} = \frac{214.63 \text{ k-in} \times 6.75 \text{ in}}{13,995.8 \text{ in}^4} = 0.104 \text{ ksi} \]

\[ CR_c = 12 \times 1.647 \text{ ksi} - 7 \times 0.104 \text{ ksi} \]

\[ CR_c = 19.04 \text{ ksi} \]
Relaxation

The relaxation loss is found using equation 9-10 of the AASHTO Standard Specifications (AASHTO 2002).

\[ CR_s = 20,000 - 0.4ES - 0.2(SH + CR_c) \]

\[ CR_s = \frac{20,000 - 0.4 \times 12,410 \text{ psi} - 0.2 \times (6,500 \text{ psi} + 19,040 \text{ psi})}{1,000 \text{ lb}} / k \]

\[ CR_s = 9.93 \text{ ksi} \]

Total Loss of Prestress

The total prestress loss is found using equation 9-3 of the AASHTO Standard Specifications (AASHTO 2002).

\[ \Delta f_s = SH + ES + CR_c + CR_s \]

\[ \Delta f_s = 6.5 \text{ ksi} + 12.41 \text{ ksi} + 19.04 \text{ ksi} + 9.93 \text{ ksi} \]

\[ \Delta f_s = 47.88 \text{ ksi} \]

Effective Prestress = 0.70 \times 270 \text{ ksi} - 47.88 \text{ ksi} = 141.12 \text{ ksi}

\[ \text{Effective Prestress} = \frac{141.12 \text{ ksi}}{270 \text{ ksi}} = 0.52 f_{pu} \]

\[ \text{Prestress Loss} = \frac{\Delta f_{pT}}{0.70 f_{pu}} = \frac{141.12 \text{ ksi}}{0.70 \times 270 \text{ ksi}} \times 100 = 25.33\% \]

\[ \text{Strand Prestrain} = \frac{141.12 \text{ ksi}}{28,900 \text{ ksi}} = 0.00488 \]
The prestress loss calculated by the provisions of the PCI Design Handbook is presented over the following pages (PCI 2004).

**Appendix E1.3**

**PCI Design Handbook Prestress Loss**

The elastic shortening loss can be found using equation 4.7.3.2 of the PCI Design Handbook (PCI 2004).

\[
ES = \frac{K_{es}E_{psf_{cir}}}{E_{ci}}
\]

\[
K_{es} = 1.0
\]

\[
E_{ps} = 28,900 \text{ ksi}
\]

\[
E_{ci} = \frac{(150 \text{ pcf})^{1.5} \times 33 \times \sqrt{4,000 \text{ ksi}}}{1000 \frac{\text{lb}}{\text{k}}}
\]

\[
A_g = 430 \text{ in}^2
\]

\[
l_g = 13,995.8 \text{ in}^4
\]

\[
K_{cir} = 0.9
\]

\[
P_i = 14 \text{ strands} \times 0.153 \frac{\text{in}^2}{\text{strand}} \times 0.7 \times 270 \text{ ksi} = 404.8 \text{ ksi}
\]

\[
e = \frac{17 \text{ in}}{2} - 1.5 \text{ in} - \frac{0.5 \text{ in}}{2} = 6.75 \text{ in}
\]

\[
\text{Diaphragm wt.} = (18 \text{ in} + 12 \text{ in}) \times (36 \text{ in} - 10 \text{ in}) \times (17\text{in} - 10\text{in}) \times \frac{0.150 \text{ kcf}}{12^3 \frac{\text{in}^3}{\text{ft}^3}}
\]
Diaphragm wt. = 0.474 \frac{k}{\text{diaphragm}}

M_{\text{diaphragm}} = \frac{PL}{4} = 0.474 \frac{k}{\text{diaphragm}} \times \left(2 \times 165 \text{ in} + 6 \text{ ft} \times \frac{12 \text{ in}}{\text{ft}}\right)

M_{\text{diaphragm}} = 47.63 \text{ k-in}

M_{\text{beam}} = \frac{wL^2}{8} = \frac{0.150 \text{ kcf} \times 430 \text{ in}^2 \times \left(2 \times 165 \text{ in} + 6 \text{ ft} \times \frac{12 \text{ in}}{\text{ft}}\right)^2}{12^3 \text{ in}^3 \frac{\text{ft}^3}{\text{ft}^3}}

M_{\text{beam}} = 754.01 \text{ k-in}

M_g = M_{\text{diaphragm}} + M_{\text{beam}} = 47.63 \text{ k-in} + 754.01 \text{ k-in} = 801.64 \text{ k-in}

f_{\text{cir}} = K_{\text{cir}} \left(\frac{P_l}{A_g} + \frac{P_l e^2}{l_g}\right) - \frac{M_g e}{l_g}

f_{\text{cir}} = 0.9 \times \left(\frac{404.8 \text{ ksi}}{430 \text{ in}^2} + \frac{404.8 \text{ ksi} \times (6.75 \text{ in})^2}{13,995.8 \text{ in}^4}\right) - \frac{801.64 \text{ k-in} \times 6.75 \text{ in}}{13,995.8 \text{ in}^4}

f_{\text{cir}} = 1.647 \text{ ksi}

ES = \frac{1.0 \times 28,900 \text{ ksi} \times 1.647 \text{ ksi}}{3,834 \text{ ksi}} = 12.41 \text{ ksi}

Creep

The creep loss is found using equation 4.7.3.4 of the PCI Design Handbook (PCI 2004).

CR = K_{cr} \left(\frac{E_p}{E_c}\right) (f_{\text{cir}} - f_{\text{cas}})

Where:

K_{cr} = 2.0

E_c = 5,500 \text{ ksi}
Barrier wt. = 2 \times \left( 90 \frac{lbs}{ft} \times \frac{k}{1000 \, lb} \times \frac{ft}{12 \, in} \right) \times \frac{1}{12 \, beams} = 0.00125 \frac{k}{in}

M_{\text{barrier}} = \frac{wL^2}{8} = \frac{0.00125 \frac{k}{in} \times \left( 2 \times 165 \, in + 6 \, ft \times \frac{12 \, in}{ft} \right)^2}{8} = 25.25 \, k \cdot in

Wearing surface wt. = 3 \, in \times \frac{0.150 \, kcf}{12^3 \frac{in^3}{ft^3}} \times 36 \, ft \times \frac{12 \, in}{ft} \times \frac{1}{12 \, beams}

Wearing surface wt. = 0.009375 \frac{k}{in}

M_{\text{wearing surface}} = \frac{wL^2}{8} = \frac{0.009375 \frac{k}{in} \times \left( 2 \times 165 \, in + 6 \, ft \times \frac{12 \, in}{ft} \right)^2}{8}

M_{\text{wearing surface}} = 189.38 \, k \cdot in

M_{sd} = M_{\text{barrier}} + M_{\text{wearing surface}} = 25.25 \, k \cdot in + 189.38 \, k \cdot in = 214.63 \, k \cdot in

f_{cas} = \frac{M_{sd}e}{I_g} = \frac{214.63 \, k \cdot in \times 6.75 \, in}{13,995.8 \, in^4} = 0.104 \, ksi

CR = 2.0 \times \left( \frac{28,900 \, ksi}{5,500 \, ksi} \right) \times (1.647 \, ksi - 0.104 \, ksi) = 16.22 \, ksi

Shrinkage

The shrinkage loss is found using equation 4.7.3.6 of the PCI Design Handbook (PCI 2004).

\[ SH = (8.2 \times 10^{-6})K_{sh}E_{ps} \left( 1 - 0.06 \frac{V}{S} \right) (100 - RH) \]

Where:

\[ K_{sh} = 1.0 \]

\[ V = A_g = 430 \, in^2 \]
\[ S = P = 2 \times (36 \text{ in} + 17 \text{ in}) + 2 \times (26 \text{ in} + 7 \text{ in}) = 172 \text{ in} \]

\[ RH = 70\% \]

\[ SH = (8.2 \times 10^{-6}) \times 1.0 \times 28,900 \text{ ksi} \times \left( 1 - 0.06 \times \frac{430 \text{ in}^2}{172 \text{ in}} \right) \times (100 - 70) \]

\[ SH = 6.04 \text{ ksi} \]

**Relaxation**

The relaxation loss is found using equation 4.7.3.7 of the PCI Design Handbook (PCI 2004).

\[ RE = [K_{re} - J(SH + CR + ES)]C \]

Where:

\[ K_{re} = 20,000 \text{ psi} \]

\[ J = 0.15 \]

\[ C = 1.0 \]

\[ RE = [20,000 \text{ psi} - 0.15 \times (6.04 \text{ ksi} + 16.22 \text{ ksi} + 12.41 \text{ ksi})] \times 1.0 = 14.80 \text{ ksi} \]

**Total Loss of Prestress**

The total prestress loss is found using equation 4.7.3.1 of the PCI Design Handbook (PCI 2004).

\[ TL = ES + CR + SH + RE \]

\[ TL = 12.41 \text{ ksi} + 16.22 \text{ ksi} + 6.04 \text{ ksi} + 14.80 \text{ ksi} \]

\[ TL = 49.47 \text{ ksi} \]
Effective Prestress \(= 0.70 \times 270 \text{ ksi} - 49.47 \text{ ksi} = 139.53 \text{ ksi}\)

\[
\text{Effective Prestress} = \frac{139.53 \text{ ksi}}{270 \text{ ksi}} = 0.52f_{pu}
\]

\[
\text{Prestress Loss} = \frac{\Delta f_{PT}}{0.70f_{pu}} = \frac{139.53 \text{ ksi}}{0.70 \times 270 \text{ ksi}} \times 100 = 26.18\% 
\]

\[
\text{Strand Prestrain} = \frac{139.53 \text{ ksi}}{28,900 \text{ ksi}} = 0.00483 
\]
Appendix E2

AASHTO Standard Specifications Ultimate Capacity

The ultimate capacity for each beam was found using the AASHTO Standard Specifications equation 9-17 (AASHTO 2002).

\[ f_{ps} = f_{pu} \left( 1 - \frac{\gamma_p}{\beta_1} \left( \frac{f_{pu}}{f_c} \right) \right) \]

An example calculation is shown for each beam in the following pages. These calculations have the number of effective strands based on the method of discounting strands adjacent to deteriorated strands. The dead load of the beam was then subtracted from the ultimate loading to be compared with the live load testing results.

The dead loading was calculated as follows:

\[ M_{DL} = \frac{430 \text{ in}^2}{144 \text{ in}^2} \times \frac{\left( 0.150 \frac{k}{ft^3} \right) \times (33.5 \text{ ft})^2}{8} = 62.8 \text{ k-ft} \]

\[ P_{DL} = \frac{2 \times 62.8 \text{ k-ft}}{33.5 \text{ ft}} = 3.75 \text{ kips} \]
Ultimate Capacity for Beam 16 with 9 strands discounted

\[ f_{ps} = f_{pu} \left( 1 - \frac{\gamma_p}{\beta_1} \left( \rho_p \frac{f_{pu}}{f'_c} \right) \right) \]

\[ A_s = (14 \text{ strands} - 9 \text{ strands}) \times 0.153 \frac{\text{in}^2}{\text{strand}} = 0.765 \text{ in}^2 \]

\[ \rho_p = \frac{A_s}{b \times d} = \frac{0.765 \text{ in}^2}{36 \text{ in} \times 15.25 \text{ in}} = 0.001393 \]

\[ f_{ps} = 270 \text{ ksi} \times \left( 1 - \frac{0.4}{0.65} \times \left( 0.001393 \times \frac{270 \text{ ksi}}{9.767 \text{ ksi}} \right) \right) = 263.60 \text{ ksi} \]

\[ a = \frac{A_s f_{ps}}{0.85 f'_c b} = \frac{0.765 \text{ in}^2 \times 263.60 \text{ ksi}}{0.85 \times 9.767 \text{ ksi} \times 36 \text{ in}} = 0.675 \text{ in} < 5 \text{ in flange} \quad \therefore \text{OK} \]

\[ \frac{\rho_p f_{ps}}{f'_c} < 0.36 \beta_1 \]

\[ 0.001393 \times 263.60 \frac{\text{ksi}}{9.767 \text{ ksi}} = 0.038 < 0.36 \times .65 = 0.234 \quad \therefore \text{OK} \]

\[ \phi = 1.0 \quad \text{for factory produced precast prestressed concrete members} \]

\[ \phi M_n = \phi A_s f_{ps} d \left( 1 - 0.6 \frac{\rho_p f_{ps}}{f'_c} \right) \]

\[ \phi M_n = 1.0 \times 0.765 \text{ in}^2 \times 263.60 \text{ ksi} \times 15.25 \text{ in} \times \left( 1 - 0.6 \times \frac{0.001393 \times 263.60 \text{ ksi}}{9.767 \text{ ksi}} \right) \]

\[ \phi M_n = 3,005.8 \text{ k-in} \]

\[ P = \frac{\phi M_n}{L} = \frac{3,005.8 \text{ k-in}}{165 \text{ in}} = 18.22 \text{ kips/point} \]

\[ P = 2 \text{ points} \times 18.22 \text{ kips/point} = 36.4 \text{ kips} \]

Subtracting out the dead load

\[ P = 36.4 \text{ kips} - 3.75 \text{ kips} = 32.7 \text{ kips} \]
Ultimate Capacity for Beam 14 with 1 strand discounted

\[ f_{ps} = f_{pu} \left( 1 - \frac{\gamma_p}{\beta_1} \left( \rho_p \frac{f_{pu}}{f_c'} \right) \right) \]

\[ A_s = (14 \text{ strands} - 1 \text{ strands}) \times 0.153 \frac{in^2}{strand} = 1.989 \text{ in}^2 \]

\[ \rho_p = \frac{A_s}{b \times d} = \frac{1.989 \text{ in}^2}{36 \text{ in} \times 15.25 \text{ in}} = 0.003623 \]

\[ f_{ps} = 270 \text{ ksi} \times \left( 1 - \frac{0.4}{0.65} \times \left( \frac{0.003623 \times 270 \text{ ksi}}{8.221 \text{ ksi}} \right) \right) = 250.23 \text{ ksi} \]

\[ a = \frac{A_s f_{ps}}{0.85 f_c' b} = \frac{1.989 \text{ in}^2 \times 250.23 \text{ ksi}}{0.85 \times 8.221 \text{ ksi} \times 36 \text{ in}} = 1.978 \text{ in} < 5 \text{ in flange} \therefore \text{OK} \]

\[ \frac{\rho_p f_{ps}}{f_c'} < 0.36 \beta_1 \]

\[ \frac{0.003623 \times 250.23 \text{ ksi}}{8.221 \text{ ksi}} = 0.110 < 0.36 \times .65 = 0.234 \therefore \text{OK} \]

\[ \phi = 1.0 \text{ for factory produced precast prestressed concrete members} \]

\[ \phi M_n = \phi A_s f_{ps} d \left( 1 - 0.6 \frac{\rho_p f_{ps}}{f_c'} \right) \]

\[ \phi M_n = 1.0 \times 1.989 \text{ in}^2 \times 250.23 \text{ ksi} \times 15.25 \text{ in} \times \left( 1 - 0.6 \times \frac{0.003623 \times 250.23 \text{ ksi}}{8.221 \text{ ksi}} \right) \]

\[ \phi M_n = 7,087.8 \text{ k- in} \]

\[ P = \frac{\phi M_n}{L} = \frac{7,087.8 \text{ k- in}}{165 \text{ in}} = 42.96 \text{ kips/point} \]

\[ P = 2 \text{ points} \times 42.96 \text{ kips/point} = 85.9 \text{ kips} \]

Subtracting out the dead load

\[ P = 85.9 \text{ kips} - 3.75 \text{ kips} = 82.2 \text{ kips} \]
Ultimate Capacity for Beam 17 with 7 strands discounted

\[ f_{ps} = f_{pu} \left( 1 - \frac{\gamma_p}{\beta_1} \left( \rho_p \frac{f_{pu}}{f'c} \right) \right) \]

\[ A_s = (14 \text{ strands} - 7 \text{ strands}) \times 0.153 \frac{\text{in}^2}{\text{strand}} = 1.071 \text{ in}^2 \]

\[ \rho_p = \frac{A_s}{b \times d} = \frac{1.071 \text{ in}^2}{36 \text{ in} \times 15.25 \text{ in}} = 0.001951 \]

\[ f_{ps} = 270 \text{ ksi} \times \left( 1 - \frac{0.4}{0.65} \times \left( 0.001951 \times \frac{270 \text{ ksi}}{8.712 \text{ ksi}} \right) \right) = 259.95 \text{ ksi} \]

\[ a = \frac{A_s f_{ps}}{0.85 f'c b} = \frac{1.071 \text{ in}^2 \times 259.95 \text{ ksi}}{0.85 \times 8.712 \text{ ksi} \times 36 \text{ in}} = 1.044 \text{ in} < 5 \text{ in flange} \therefore \text{OK} \]

\[ \frac{\rho_p f_{ps}}{f'c} < 0.36\beta_1 \]

\[ \frac{0.001951 \times 259.95 \text{ ksi}}{8.712 \text{ ksi}} = 0.058 < 0.36 \times 0.65 = 0.234 \therefore \text{OK} \]

\[ \phi = 1.0 \text{ for factory produced precast prestressed concrete members} \]

\[ \phi M_n = \phi A_s f_{ps} d \left( 1 - 0.6 \frac{\rho_p f_{ps}}{f'c} \right) \]

\[ \phi M_n = 1.0 \times 1.071 \text{ in}^2 \times 259.95 \text{ ksi} \times 15.25 \text{ in} \times \left( 1 - 0.6 \times \frac{0.001951 \times 259.95 \text{ ksi}}{8.712 \text{ ksi}} \right) \]

\[ \phi M_n = 4,097.5 \text{ k- in} \]

\[ p = \frac{\phi M_n}{L} = \frac{4,097.5 \text{ k- in}}{165 \text{ in}} = 24.83 \frac{\text{kips}}{\text{point}} \]

\[ P = 2 \text{ points} \times 24.83 \frac{\text{kips}}{\text{point}} = 49.7 \text{ kips} \]

Subtracting out the dead load

\[ P = 49.7 \text{ kips} - 3.75 \text{ kips} = 45.9 \text{ kips} \]
Ultimate Capacity for Beam 19 with 8 strands discounted

\[ f_{ps} = f_{pu} \left( 1 - \frac{\gamma_p}{\beta_1} \left( \frac{\rho_p}{f_c'} \right) \right) \]

\[ A_s = (14 \text{ strands} - 8 \text{ strands}) \times 0.153 \frac{\text{in}^2}{\text{strand}} = 0.918 \text{ in}^2 \]

\[ \rho_p = \frac{A_s}{b \times d} = \frac{0.918 \text{ in}^2}{36 \text{ in} \times 15.25 \text{ in}} = 0.001672 \]

\[ f_{ps} = 270 \text{ ksi} \times \left( 1 - \frac{0.4}{0.65} \times \left( 0.001672 \times \frac{270 \text{ ksi}}{10.148 \text{ ksi}} \right) \right) = 262.61 \text{ ksi} \]

\[ a = \frac{A_s f_{ps}}{0.85 f_c' b} = \frac{0.918 \text{ in}^2 \times 262.61 \text{ ksi}}{0.85 \times 10.148 \text{ ksi} \times 36 \text{ in}} = 0.776 \text{ in} < 5 \text{ in flange} \therefore \text{OK} \]

\[ \frac{\rho_p f_{ps}}{f_c'} < 0.36 \beta_1 \]

\[ 0.001672 \times 262.61 \text{ ksi} \]

\[ 10.148 \text{ ksi} = 0.043 < 0.36 \times 0.65 = 0.234 \therefore \text{OK} \]

\[ \phi = 1.0 \text{ for factory produced precast prestressed concrete members} \]

\[ \phi M_n = \phi A_s f_{ps} d \left( 1 - 0.6 \frac{\rho_p f_{ps}}{f_c'} \right) \]

\[ \phi M_n = 1.0 \times 0.918 \text{ in}^2 \times 262.61 \text{ ksi} \times 15.25 \text{ in} \times \left( 1 - 0.6 \times \frac{0.001672 \times 262.61 \text{ ksi}}{10.148 \text{ ksi}} \right) \]

\[ \phi M_n = 3,580.9 \text{ k- in} \]

\[ P = \frac{\phi M_n}{L} = \frac{3,580.9 \text{ k- in}}{165 \text{ in}} = 21.70 \text{ kips/point} \]

\[ P = 2 \text{ points} \times 21.70 \text{ kips/point} = 43.4 \text{ kips} \]

Subtracting out the dead load

\[ P = 43.4 \text{ kips} - 3.75 \text{ kips} = 39.7 \text{ kips} \]
Appendix E3

Effective Prestressing Force

The effective prestress force for each beam was calculated using the basic equation of prestressing. This equation can be rearranged to solve for the effective prestress force.

\[
\frac{P}{A} + \frac{Pe_y}{I} - \frac{M_{sw}y}{I} - \frac{M_{app}y}{I} = 0
\]

\[\therefore P = \frac{M_{sw}y + M_{app}y}{1 + \frac{e_y}{I}}\]

The applied moment was found by taking the loading required to open the cracks. This loading was found from where two lines following the clip gauge slopes intersect on the respective clip gauge graphs. At this loading, the stress across the crack must be zero.

The number of effective prestressed strands was calculated by taking the effective prestress force for each beam divided by the effective prestress force from the provisions of the AASHTO LRFD Bridge Design Specifications (AASHTO 2007). This gave an effective area of steel, from which the number of strands was calculated. The pre-cracking stiffness of each beam was found by taking the loading at which the cracks opened and dividing by the corresponding displacement. The moment of inertia was found based on this stiffness. All of these calculations for each beam are shown in the following pages.

Appendix E3.1 – Beam 16 Supplemental Calculations

Appendix E3.2 – Beam 14 Supplemental Calculations

Appendix E3.3 – Beam 17 Supplemental Calculations

Appendix E3.4 – Beam 19 Supplemental Calculations
Appendix E3.1

Beam 16 Supplemental Calculations
Effective Prestress Force for Beam 16

From the west clip gauge:

\[ x_1 = 171 \text{ in} \]
\[ y_1 = 13.25 \text{ in} \]

\[ Loading_1 = 20 \text{ kips} \]
\[ y = 8.5 \text{ in} - (17 \text{ in} - 13.25 \text{ in}) = 4.75 \text{ in} \]

\[ M_{sw} = \frac{wx}{2} (L - x) = \frac{0.448 \frac{k}{ft} \times \frac{ft}{12 \text{ in}} \times 171 \text{ in}}{2} (402 \text{ in} - 171 \text{ in}) \]

\[ M_{sw} = 737.21 \text{ k- in} \]

Clip gauge is in the constant moment region.

\[ M_{app} = PL = \left( \frac{20 \text{ kips}}{2} \right) \times 165 \text{ in} = 1,650 \text{ k- in} \]

\[ P = \frac{737.21 \text{ k- in} \times 4.75 \text{ in}}{13,995.8 \text{ in}^4} + \frac{1,650 \text{ k- in} \times 4.75 \text{ in}}{13,995.8 \text{ in}^4} = 175.50 \text{ k} \]

From the east clip gauge:

\[ x_2 = 183 \text{ in} \]
\[ y_2 = 12.25 \text{ in} \]

\[ Loading_2 = 17.5 \text{ kips} \]
\[ y = 8.5 \text{ in} - (17 \text{ in} - 12.25 \text{ in}) = 3.75 \text{ in} \]

\[ M_{sw} = \frac{wx}{2} (L - x) = \frac{0.448 \frac{k}{ft} \times \frac{ft}{12 \text{ in}} \times 183 \text{ in}}{2} (402 \text{ in} - 183 \text{ in}) \]

\[ M_{sw} = 747.96 \text{ k- in} \]
Clip gauge is in the constant moment region.

\[ M_{app} = PL = \left( \frac{17.5 \text{ kips}}{2} \right) \times 165 \text{ in} = 1,443.75 \text{ k-in} \]

\[ P = \frac{747.96 \text{ k-in} \times 3.75 \text{ in}}{10,276.5 \text{ in}^4} + \frac{1,443.75 \text{ k-in} \times 3.75 \text{ in}}{10,276.5 \text{ in}^4} = 142.05 \text{ k} \]

**Average Effective Prestress Force for Beam 16:**

\[ P = \frac{175.50 \text{ k} + 142.05 \text{ k}}{2} = 158.8 \text{ k} \]

**Calculated Effective Strands for Beam 16:**

*AASHTO LRFD Bridge Design Specification Prestress Loss = 18.44%*

Effective Prestress = \((0.70 \times 270 \text{ ksi}) \times (1 - 0.1844) = 154.15 \text{ ksi} \)

Effective Strand Area = \(\frac{158.8 \text{ k}}{154.15 \text{ ksi}} = 1.030 \text{ in}^2 \)

Effective Strands = \(\frac{1.030 \text{ in}^2}{0.153 \text{ in}^2/\text{strand}} = 6.73 \text{ strands} \)

*Beam 14 Prestress Loss = 14.95%*

Effective Prestress = \((0.70 \times 270 \text{ ksi}) \times (1 - 0.1495) = 160.75 \text{ ksi} \)

Strand Prestrain = \(\frac{160.75 \text{ ksi}}{28,900 \text{ ksi}} = 0.00556 \)

Effective Strand Area = \(\frac{158.8 \text{ k}}{160.75 \text{ ksi}} = 0.988 \text{ in}^2 \)

Effective Strands = \(\frac{0.988 \text{ in}^2}{0.153 \text{ in}^2/\text{strand}} = 6.46 \text{ strands} \)
Pre-Cracking Stiffness for Beam 16:

West Loading = 20 kips

West Wire Potentiometer Deflection = 0.5978 in

West Stiffness = \( \frac{20 \text{ kips}}{0.5978 \text{ in}} = 33.46 \text{ kips/in} \)

East Loading = 17.5 kips

East Wire Potentiometer Deflection = 0.5291 in

East Stiffness = \( \frac{17.5 \text{ kips}}{0.5291 \text{ in}} = 33.08 \text{ kips/in} \)

Average Pre-Cracking Stiffness for Beam 16:

\[ K = \frac{33.46 \text{ kips/in} + 33.08 \text{ kips/in}}{2} = 33.3 \text{ kips/in} \]

Modulus of Elasticity and Moment of Inertia for Beam 16:

\[ E = 57,000 \sqrt{f_c'} = \frac{57,000 \sqrt{9,767 \text{ psi}}}{1,000} = 5,633 \text{ ksi} \]

\[ \Delta = \frac{Pa}{24EI} (3l^2 - 4a^2) + \frac{5wl^4}{384EI} \quad \therefore I = \frac{Pa}{24E\Delta} (3l^2 - 4a^2) + \frac{5wl^4}{384E\Delta} \]

\[ I = \frac{(20 k + 17.5 k) \times 2 \times 2}{2 \times 2} \times 165 \text{ in} \times (3 \times (402 \text{ in})^2 - 4 \times (165 \text{ in})^2) \]

\[ \frac{24 	imes 5,633 \text{ ksi} \times \left( \frac{0.5978 \text{ in} \times 0.5291 \text{ in}}{2} \right)}{2} \]

\[ + \frac{5 \times 0.037 \times k}{384 \times 5,633 \text{ ksi} \times \left( \frac{0.5978 \text{ in} \times 0.5291 \text{ in}}{2} \right)} \]

\[ I = 11,597 \text{ in}^4 \]

207
Appendix E3.2

Beam 14 Supplemental Calculations
Effective Prestress Force for Beam 14

From the west clip gauge:

\[ x_1 = 173.5 \text{ in} \]
\[ y_1 = 15.5 \text{ in} \]

\[ \text{Loading}_1 = 38 \text{ kips} \]

\[ y = 8.5 \text{ in} - (17 \text{ in} - 15.5 \text{ in}) = 7 \text{ in} \]

\[ M_{sw} = \frac{wx}{2} (L - x) = \frac{0.448 \frac{ft}{k} \times \frac{ft}{12 \text{ in}} \times 173.5 \text{ in}}{2} (402 \text{ in} - 173.5 \text{ in}) \]

\[ M_{sw} = 739.9 \text{ k-in} \]

Clip gauge is in the constant moment region.

\[ M_{app} = PL = \left( \frac{38 \text{ kips}}{2} \right) \times 165 \text{ in} = 3,135 \text{ k-in} \]

\[ P = \frac{739.9 \text{ k-in} \times 7 \text{ in}}{13,995.8 \text{ in}^4} + \frac{3,135 \text{ k-in} \times 7 \text{ in}}{13,995.8 \text{ in}^4} = 339.91 \text{ k} \]

From the east clip gauge:

\[ x_2 = 169.5 \text{ in} \]
\[ y_2 = 15.25 \text{ in} \]

\[ \text{Loading}_2 = 40 \text{ kips} \]

\[ y = 8.5 \text{ in} - (17 \text{ in} - 15.25 \text{ in}) = 6.75 \text{ in} \]

\[ M_{sw} = \frac{wx}{2} (L - x) = \frac{0.448 \frac{ft}{k} \times \frac{ft}{12 \text{ in}} \times 169.5 \text{ in}}{2} (402 \text{ in} - 169.5 \text{ in}) \]

\[ M_{sw} = 735.49 \text{ k-in} \]
Clip gauge is in the constant moment region.

\[ M_{app} = PL = \left( \frac{40 \text{ kips}}{2} \right) \times 165 \text{ in} = 3,300 \text{ k-in} \]

\[ P = \frac{735.49 \text{ k-in} \times 6.75 \text{ in}}{13,995.8 \text{ in}^4} + \frac{3,300 \text{ k-in} \times 6.75 \text{ in}}{13,995.8 \text{ in}^4} = 348.73 \text{ k} \]

**Average Effective Prestress Force for Beam 14:**

\[ P = \frac{339.91 \text{ k} + 348.73 \text{ k}}{2} = 344.3 \text{ k} \]

**Calculated Effective Strands for Beam 14:**

*AASHTO LRFD Bridge Design Specification Prestress Loss* = 18.44 %

*Effective Prestress* = \((0.70 \times 270 \text{ ksi}) \times (1 - 0.1844) = 154.15 \text{ ksi}\)

*Effective Strand Area* = \(\frac{344.3 \text{ k}}{154.15 \text{ ksi}} = 2.234 \text{ in}^2\)

*Effective Strands* = \(\frac{2.234 \text{ in}^2}{0.153 \text{ in}^2/\text{strand}} = 14.60 \text{ strands}\)

*Beam 14 Prestress Loss* = 14.95 %

*Effective Prestress* = \((0.70 \times 270 \text{ ksi}) \times (1 - 0.1495) = 160.75 \text{ ksi}\)

*Strand Prestrain* = \(\frac{160.75 \text{ ksi}}{28,900 \text{ ksi}} = 0.00556\)

*Effective Strand Area* = \(\frac{344.3 \text{ k}}{160.75 \text{ ksi}} = 2.142 \text{ in}^2\)

*Effective Strands* = \(\frac{2.142 \text{ in}^2}{0.153 \text{ in}^2/\text{strand}} = 14.00 \text{ strands}\)
Pre-Cracking Stiffness for Beam 14:

West Loading = 38 kips

West Wire Potentiometer Deflection = 0.8508 in

West Stiffness = \( \frac{38 \text{ kips}}{0.8508 \text{ in}} = 44.66 \text{ kips/in} \)

East Loading = 40 kips

East Wire Potentiometer Deflection = 0.9122 in

East Stiffness = \( \frac{40 \text{ kips}}{0.9122 \text{ in}} = 43.85 \text{ kips/in} \)

Average Pre-Cracking Stiffness for Beam 14:

\[ K = \frac{44.66 \text{ kips/in} + 43.85 \text{ kips/in}}{2} = 44.3 \text{ kips/in} \]

Modulus of Elasticity and Moment of Inertia for Beam 14:

\( E = 57,000 \sqrt{f_c} = \frac{57,000 \sqrt{8,221 \text{ psi}}}{1,000} = 5,168 \text{ ksi} \)

\[ \Delta = \frac{Pa}{24EI} (3l^2 - 4a^2) + \frac{5wl^4}{384EI} \quad \therefore \quad I = \frac{Pa}{24E\Delta} (3l^2 - 4a^2) + \frac{5wl^4}{384E\Delta} \]

\[ I = \frac{(38 k + 40 k)}{2 \times 2} \times 165 \text{ in} \times (3 \times (402 \text{ in})^2 - 4 \times (165 \text{ in})^2) \]

\[ 24 \times 5,168 \text{ ksi} \times \left( \frac{0.8508 \text{ in} \times 0.9122 \text{ in}}{2} \right) \]

\[ + \frac{5 \times 0.037}{384 \times 5,168 \text{ ksi} \times \left( \frac{0.8508 \text{ in} \times 0.9122 \text{ in}}{2} \right)} \times (402 \text{ in})^4 \]

\[ I = 13,824 \text{ in}^4 \]
Appendix E3.3

Beam 17 Supplemental Calculations
Effective Prestress Force for Beam 17

From the southeast clip gauge:

\[ x_1 = 173.75 \text{ in} \]
\[ y_1 = 15.75 \text{ in} \]

\[ Loading_1 = 27.5 \text{ kips} \]

\[ y = 8.5 \text{ in} - (17 \text{ in} - 15.75 \text{ in}) = 7.25 \text{ in} \]

\[ M_{sw} = \frac{wx}{2} (L - x) = \frac{0.448 k \frac{ft}{12 \text{ in}} \times \frac{ft}{2} \times 173.75 \text{ in}}{2} (402 \text{ in} - 173.75 \text{ in}) \]

\[ M_{sw} = 740.15 \text{ k-in} \]

Clip gauge is in the constant moment region.

\[ M_{app} = PL = \left( \frac{27.5 \text{ kips}}{2} \right) \times 165 \text{ in} = 2,268.75 \text{ k-in} \]

\[ P = \frac{740.15 \text{ k-in} \times 7.25 \text{ in}}{13,995.8 \text{ in}^4} + \frac{2,268.75 \text{ k-in} \times 7.25 \text{ in}}{13,995.8 \text{ in}^4} = 267.71 \text{ k} \]

From the northeast clip gauge:

\[ x_2 = 169 \text{ in} \]
\[ y_2 = 12.75 \text{ in} \]

\[ Loading_2 = 27 \text{ kips} \]

\[ y = 8.5 \text{ in} - (17 \text{ in} - 12.75 \text{ in}) = 4.25 \text{ in} \]

\[ M_{sw} = \frac{wx}{2} (L - x) = \frac{0.448 k \frac{ft}{12 \text{ in}} \times \frac{ft}{2} \times 169 \text{ in}}{2} (402 \text{ in} - 169 \text{ in}) \]

\[ M_{sw} = 734.90 \text{ k-in} \]
Clip gauge is in the constant moment region.

\[ M_{app} = PL = \left( \frac{27 \text{ kips}}{2} \right) \times 165 \text{ in} = 2,227.5 \text{ k-in} \]

\[ P = \frac{734.90 \text{ k-in} \times 4.25 \text{ in} + 2,227.5 \text{ k-in} \times 4.25 \text{ in}}{13,995.8 \text{ in}^4} + \frac{1,750 \text{ in} \times 4.25 \text{ in}}{13,995.8 \text{ in}^4} = 205.60 \text{ k} \]

**Average Effective Prestress Force for Beam 17:**

\[ P = \frac{267.71 \text{ k} + 205.60 \text{ k}}{2} = 236.7 \text{ k} \]

**Calculated Effective Strands for Beam 17:**

**AASHTO LRFD Bridge Design Specification Prestress Loss** = 18.44 %

**Effective Prestress** = \((0.70 \times 270 \text{ ksi}) \times (1 - 0.1844) = 154.15 \text{ ksi}\)

**Effective Strand Area** = \(\frac{236.7 \text{ k}}{154.15 \text{ ksi}} = 1.535 \text{ in}^2\)

**Effective Strands** = \(\frac{1.535 \text{ in}^2}{0.153 \text{ in}^2/\text{strand}} = 10.03 \text{ strands}\)

**Beam 14 Prestress Loss** = 14.95 %

**Effective Prestress** = \((0.70 \times 270 \text{ ksi}) \times (1 - 0.1495) = 160.75 \text{ ksi}\)

**Strand Prestrain** = \(\frac{160.75 \text{ ksi}}{28,900 \text{ ksi}} = 0.00556\)

**Effective Strand Area** = \(\frac{236.7 \text{ k}}{160.75 \text{ ksi}} = 1.472 \text{ in}^2\)

**Effective Strands** = \(\frac{1.472 \text{ in}^2}{0.153 \text{ in}^2/\text{strand}} = 9.62 \text{ strands}\)
Pre-Cracking Stiffness for Beam 17:

East Loading = 27.5 kips

East Wire Potentiometer Deflection = 0.9074 in

East Stiffness = \( \frac{27.5 \text{ kips}}{0.9074 \text{ in}} = 30.31 \text{ kips/in} \)

East Loading = 27 kips

East Wire Potentiometer Deflection = 0.8881 in

East Stiffness = \( \frac{27 \text{ kips}}{0.8881 \text{ in}} = 30.40 \text{ kips/in} \)

Average Pre-Cracking Stiffness for Beam 17:

\[
K = \frac{30.31 \text{ kips/in} + 30.40 \text{ kips/in}}{2} = 30.4 \text{ kips/in}
\]

Modulus of Elasticity and Moment of Inertia for Beam 17:

\[
E = 57,000 \sqrt{f_c} = \frac{57,000 \sqrt{8,712 \text{ psi}}}{1,000} = 5,320 \text{ ksi}
\]

\[
\Delta = \frac{Pa}{24EI} (3l^2 - 4a^2) + \frac{5wl^4}{384EI} \quad \therefore I = \frac{Pa}{24E\Delta} (3l^2 - 4a^2) + \frac{5wl^4}{384E\Delta}
\]

\[
l = \frac{(27.5 k + 27 k)}{2 \times 2} \times 165 \text{ in} \times (3 \times (402 \text{ in})^2 - 4 \times (165 \text{ in})^2)
\]

\[
\begin{align*}
24 \times 5,320 \text{ ksi} \times \frac{(0.9074 \text{ in} \times 0.8881 \text{ in})}{2} + \\
5 \times 0.037 \frac{k}{in} \times (402 \text{ in})^4
\end{align*}
\]

\[
I = 10,007 \text{ in}^4
\]
Appendix E3.4

Beam 19 Supplemental Calculations
Effective Prestress Force for Beam 19

From the southwest clip gauge:

\[ x_1 = 135 \text{ in} \]
\[ y_1 = 13.5 \text{ in} \]

\( Loading_1 = 31.5 \text{ kips} \)

\[ y = 8.5 \text{ in} - (17 \text{ in} - 13.5 \text{ in}) = 5 \text{ in} \]

\[ M_{sw} = \frac{wx}{2} (L - x) = \frac{0.448 \frac{k}{ft} \times \frac{ft}{12 \text{ in}} \times 135 \text{ in}}{2} (402 \text{ in} - 135 \text{ in}) \]

\[ M_{sw} = 672.71 \text{ k-in} \]

Clip gauge is not in the constant moment region.

\[ M_{app} = PL = \left( \frac{31.5 \text{ kips}}{2} \right) \times 135 \text{ in} = 2,126.25 \text{ k-in} \]

\[ P = \frac{672.71 \text{ k-in} \times 5 \text{ in}}{13,995.8 \text{ in}^4} + \frac{2,126.25 \text{ k-in} \times 5 \text{ in}}{13,995.8 \text{ in}^4} = 211.09 \text{ k} \]

From the northwest clip gauge:

\[ x_2 = 184.75 \text{ in} \]
\[ y_2 = 12.75 \text{ in} \]

\( Loading_2 = 27 \text{ kips} \)

\[ y = 8.5 \text{ in} - (17 \text{ in} - 12.75 \text{ in}) = 4.25 \text{ in} \]

\[ M_{sw} = \frac{wx}{2} (L - x) = \frac{0.448 \frac{k}{ft} \times \frac{ft}{12 \text{ in}} \times 184.75 \text{ in}}{2} (402 \text{ in} - 184.75 \text{ in}) \]

\[ M_{sw} = 749.08 \text{ k-in} \]
Clip gauge is in the constant moment region.

\[ M_{app} = PL = \left( \frac{27 \text{ kips}}{2} \right) \times 165 \text{ in} = 2,227.5 \text{ k-in} \]

\[ P = \frac{749.08 \text{ k-in} \times 4.25 \text{ in}}{13,995.8 \text{ in}^4} + \frac{2,227.5 \text{ k-in} \times 4.25 \text{ in}}{13,995.8 \text{ in}^4} = 206.59 \text{ k} \]

**Average Effective Prestress Force for Beam 19:**

\[ P = \frac{211.09 \text{ k} + 206.59 \text{ k}}{2} = 208.8 \text{ k} \]

**Calculated Effective Strands for Beam 19:**

*AASHTO LRFD Bridge Design Specification Prestress Loss* = 18.44%

*Effective Prestress* = \((0.70 \times 270 \text{ ksi}) \times (1 - 0.1844) = 154.15 \text{ ksi}\)

*Effective Strand Area* = \(\frac{208.8 \text{ k}}{154.15 \text{ ksi}} = 1.355 \text{ in}^2\)

*Effective Strands* = \(\frac{1.355 \text{ in}^2}{0.153 \text{ in}^2/\text{strand}} = 8.85 \text{ strands}\)

*Beam 14 Prestress Loss* = 14.95%

*Effective Prestress* = \((0.70 \times 270 \text{ ksi}) \times (1 - 0.1495) = 160.75 \text{ ksi}\)

*Strand Prestrain* = \(\frac{160.75 \text{ ksi}}{28,900 \text{ ksi}} = 0.00556\)

*Effective Strand Area* = \(\frac{208.8 \text{ k}}{160.75 \text{ ksi}} = 1.299 \text{ in}^2\)

*Effective Strands* = \(\frac{1.299 \text{ in}^2}{0.153 \text{ in}^2/\text{strand}} = 8.49 \text{ strands}\)
Pre-Cracking Stiffness for Beam 19:

*West Loading* = 31.5 kips

*West Wire Potentiometer Deflection* = 0.7444 in

*West Stiffness* = \( \frac{31.5 \text{ kips}}{0.7444 \text{ in}} = 42.32 \text{ kips/in} \)

*West Loading* = 27 kips

*West Wire Potentiometer Deflection* = 0.9041 in

*West Stiffness* = \( \frac{27 \text{ kips}}{0.9041 \text{ in}} = 29.86 \text{ kips/in} \)

**Average Pre-Cracking Stiffness for Beam 19:**

\[ K = \frac{42.32 \text{ kips/in} + 29.86 \text{ kips/in}}{2} = 36.1 \text{ kips/in} \]

**Modulus of Elasticity and Moment of Inertia for Beam 19:**

\[ E = 57,000 \sqrt{f_c} = \frac{57,000 \sqrt{10,148 \text{ psi}}}{1,000} = 5,742 \text{ ksi} \]

\[ I = \frac{Pa}{24EI} (3l^2 - 4a^2) + \frac{5wl^4}{384EI} \]

\[ \Delta = \frac{Pa}{24E} \frac{(3l^2 - 4a^2)}{l} + \frac{5wl^4}{384EI} \]

\[ I = \frac{24 \times 5,742 \text{ ksi} \times \left( \frac{0.7444 \text{ in} \times 0.9041 \text{ in}}{2} \right)}{2 \times 2} \times 165 \text{ in} \times (3 \times (402 \text{ in})^2 - 4 \times (165 \text{ in})^2) \]

\[ I = \frac{5 \times 0.037 \frac{k}{m} \times (402 \text{ in})^4}{384 \times 5,742 \text{ ksi} \times \left( \frac{0.7444 \text{ in} \times 0.9041 \text{ in}}{2} \right)} \]

\[ I = 10,644 \text{ in}^4 \]
Appendix E4

Middle Span Load Rating

The load rating was determined following the Load Factor Rating (LFR) method. The load rating was determined using equation 6B.5.1-1 from The Manual for Bridge Evaluation (AASHTO 2008).

\[
RF = \frac{C - A_1 D}{A_2 L(1 + I)}
\]

Where:

\( A_1 = 1.3 \)

\( A_2 = 1.3 \) for Operating

\( A_2 = 2.17 \) for Inventory

\( C = Member \ Capacity \)

\( D = Dead \ Load \ Effect \)

\( L = Live \ Load \ Effect \)

The following sample calculation is for beam 18, which had the most visible deteriorated prestressed strands of all the middle span beams. Therefore, this beam has the lowest ultimate loading and the lowest load rating of all the middle span beams. The results for each of the middle span beams are presented in Table 9.2.1. The measured concrete core strength was used for the middle span beams when known. For the other middle span beams where the actual strength was unknown, concrete core strength of 8,000 psi was used (Gulistani 2010).
Live Load Distribution Factor

\[ N_L = \frac{36 \text{ ft}}{12 \text{ ft/lane}} = 3 \text{ lanes} \]

\[ \mu = 0.2 \]

\[ J = \frac{2tt_f(b-t)(h-t_f)^2}{bt + htf - t^2 - t_f^2} = \frac{2 \times 5 \text{ in} \times 5 \text{ in} \times (36 \text{ in} - 5 \text{ in})^2 \times (17 \text{ in} - 5 \text{ in})^2}{36 \text{ in} \times 5 \text{ in} + 17 \text{ in} \times 5 \text{ in} - (5 \text{ in})^2 - (5 \text{ in})^2} \]

\[ J = 32,182.3 \text{ in}^4 \]

\[ l = 13,995.8 \text{ in}^4 \]

\[ \frac{l}{J} = \frac{13,995.8 \text{ in}^4}{32,182.3 \text{ in}^4} = 0.435 \]

\[ \sqrt{\frac{l}{J}} = \sqrt{0.435} = 0.659 < 5.0 \therefore \text{OK} \]

\[ K = \sqrt{(1 + \mu)\left(\frac{l}{J}\right)} = \sqrt{(1 + 0.2)(0.435)} = 0.722 \]

\[ \frac{W}{L} = \frac{36 \text{ ft}}{37.5 \text{ ft}} = 0.960 \]

\[ C = K\left(\frac{W}{L}\right) = 0.722 \times 0.960 = 0.694 \]

\[ S = 3 \text{ ft} \]

\[ D = (5.75 - 0.5N_L) + 0.7N_L(1 - 0.2C)^2 \]

\[ D = (5.75 - 0.5 \times 3) + 0.7 \times 3 \times (1 - 0.2 \times 0.694)^2 = 5.808 \]

\[ DF = \text{Distribution Factor} = \frac{S}{D} = \frac{3 \text{ ft}}{5.808} = 0.517 \]

\[ \frac{\text{wheel lines}}{\text{beam}} = \frac{0.517}{5.808} = 0.258 \frac{\text{lanes}}{\text{beam}} \]

\[ I = \text{Live Load Impact Factor} = \frac{50}{L + 125} = \frac{50}{37.5 \text{ ft} + 125} = 0.308 \]
\[ M_{beam} = \frac{wL^2}{8} = \frac{0.150 \text{ kcf} \times 430 \text{ in}^2 \times \frac{ft^2}{144 \text{ in}^2} \times (37.5 \text{ ft})^2}{8} = 78.74 \text{ k-ft} \]

\[ \text{Diaphragm wt.} = (18 \text{ in} + 12 \text{ in}) \times (36 \text{ in} - 10 \text{ in}) \times (17\text{in} - 10\text{in}) \times \frac{0.150 \text{ kcf}}{12^3 \text{ in}^3 \text{ ft}^3} \]

\[ \text{Diaphragm wt.} = 0.474 \frac{k}{\text{diaphragm}} \]

\[ M_{\text{diaphragm}} = \frac{PL}{4} = \frac{0.474 \frac{k}{\text{diaphragm}} \times 37.5 \text{ ft}}{4} = 4.44 \text{ k-ft} \]

\[ M_{DL} = M_{\text{diaphragm}} + M_{\text{beam}} = 4.44 \text{ k-ft} + 78.74 \text{ k-ft} = 83.18 \text{ k-ft} \]

Following the same procedure as in Appendix E2:

\[ \text{Beam 18 Moment Capacity} = 201.9 \text{ k-ft} \]

\[ \text{Operating RF} = \frac{201.9 \text{ k-ft} - 1.3 \times (78.74 \text{ k-ft} + 4.44 \text{ k-ft})}{1.3 \times (405 \frac{k}{\text{lane}} \times 0.258 \frac{\text{lanelanes}}{\text{beam}}) \times (1 + 0.308)} = 0.53 \]

\[ \text{Inventory RF} = \frac{201.9 \text{ k-ft} - 1.3 \times (78.74 \text{ k-ft} + 4.44 \text{ k-ft})}{2.17 \times (405 \frac{k}{\text{lane}} \times 0.258 \frac{\text{lanelanes}}{\text{beam}}) \times (1 + 0.308)} = 0.32 \]
Appendix E5

Middle Span Capacity

The middle span capacity was determined using the AASHTO Standard Specifications (AASHTO 2002) ultimate loading for each of the beams. The ultimate moment for each beam was calculated following the AASHTO Standard Specifications. When the concrete compressive strength was unknown, it was assumed to be 8,000 psi (Gulistani 2010). The summarized results for each of the beams are presented in Table 9.3.1. The procedure used was the same as used in Appendix E2.

**Moment per middle span beam**

\[ M = 1.3 \left( M_{DL} + 1.67(1 + I) \left( \frac{M_{LL}}{n} \right) \right) \]

Where:

\[ n = \text{number of surviving beams} \]

\[ I = \text{Impact Factor} = \frac{50}{L + 125} = \frac{50}{37.5 \text{ ft} + 125} = 0.308 \]

\[ \text{Diaphragm wt.} = (18 \text{ in} + 12 \text{ in}) \times (36 \text{ in} - 10 \text{ in}) \times (17\text{ in} - 10\text{in}) \times \frac{0.150 \text{ kcf}}{12^3 \text{ in}^3/\text{ft}^3} \]

\[ \text{Diaphragm wt.} = 0.474 \frac{k}{\text{diaphragm}} \]

\[ M_{\text{diaphragm}} = \frac{PL}{4} = \frac{0.474 \frac{k}{\text{diaphragm}} \times (37.5 \text{ ft})}{4} = 4.44 \text{ k-ft} \]

\[ M_{\text{beam}} = \frac{wL^2}{8} = \frac{0.150 \text{ kcf} \times 430 \text{ in}^2 \times \frac{ft^2}{144 \text{ in}^2} \times (37.5 \text{ ft})^2}{8} = 78.74 \text{ k-ft} \]
\[ M_{DL} = M_{diaphragm} + M_{beam} = 4.44 \text{ k-ft} + 78.74 \text{ k-ft} = 83.18 \text{ k-ft} \]

\[ \text{Lanes} = \frac{\text{width}}{12} = \frac{36 \text{ ft}}{12 \text{ ft/lane}} = 3 \text{ lanes} \]

\[ M_{LL} = 3 \text{ lanes} \times 405 \text{ k-ft/lane} = 1,215 \text{ k-ft} \]

**Factored Loading Investigation**

Initially all twelve middle span beams are intact.

\[ M = 1.3 \left( 83.18 \text{ k-ft} + 1.67 \times (1 + 0.308) \times \left( \frac{1,215 \text{ k-ft}}{12 \text{ beams}} \right) \right) = 395.6 \text{ k-ft} \]

Five beams will fail at this moment.

Now there are seven surviving beams remaining intact.

\[ M = 1.3 \left( 83.18 \text{ k-ft} + 1.67 \times (1 + 0.308) \times \left( \frac{1,215 \text{ k-ft}}{7 \text{ beams}} \right) \right) = 600.9 \text{ k-ft} \]

Three more beams will fail at this moment, for a total of eight beams considered failed.

Now there are four surviving beams remaining intact.

\[ M = 1.3 \left( 83.18 \text{ k-ft} + 1.67 \times (1 + 0.308) \times \left( \frac{1,215 \text{ k-ft}}{4 \text{ beams}} \right) \right) = 970.5 \text{ k-ft} \]

The remaining four beams will fail at this moment. Therefore, the entire middle span would fail at the designed capacity of 405 k-ft per lane with the factored load combinations. Load restrictions would have been necessary had the bridge not been decommissioned.
**Service Loading Investigation**

Initially all twelve middle span beams are intact.

\[ M = 1.0 \left( 83.18 \text{ k-ft} + 1.0 \times (1 + 0.308) \times \left( \frac{1,215 \text{ k-ft}}{12 \text{ beams}} \right) \right) = 215.6 \text{ k-ft} \]

One beam will fail at this moment.

Now there are eleven surviving beams remaining intact.

\[ M = 1.0 \left( 83.18 \text{ k-ft} + 1.0 \times (1 + 0.308) \times \left( \frac{1,215 \text{ k-ft}}{11 \text{ beams}} \right) \right) = 227.6 \text{ k-ft} \]

No additional beams will fail at this moment. Therefore, only one beam from the middle span would fail at the designed capacity of 405 k-ft per lane. Load restrictions would have been necessary had the bridge not been decommissioned, since a middle span beam would fail.