ANALYSIS AND REHABILITATION
OF THE 1882 WROUGHT
IRON BRIDGE

by

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A THESIS

Submitted in partial fulfillment of the requirements for the degree of Master of Science in the Department of Civil, Construction, and Environmental Engineering in the Graduate School of The University of Alabama

TUSCALOOSA, ALABAMA

2010
ABSTRACT

The Friends of Historic Northport in conjunction with the City of Northport planned to relocate and preserve an existing span of a bridge that originally crossed the Black Warrior River in 1882. The 1882 Bridge is an old wrought iron arch-shaped truss bridge manufactured by the King Iron and Bridge Company of Ohio.

A tensile test and metallographic analysis were performed on samples taken from the bridge, and it was determined that the bridge is composed of at least two grades of wrought iron. The tests also indicated the presence of brittle iron. The wood decking system, the concrete bridge abutments, and the keystone retaining wall for the bridge approaches were designed for material quantities for the relocation project.

A structural analysis of the bridge at its new location showed that some of the bridge members were slightly overstressed under certain loading conditions. Parts of the top chord, floor beam, and diagonals were overstressed under the dead and pedestrian loads, and the bottom diagonals were overstressed under the wind load. Four lifting methods were analyzed for the bridge’s relocation. The method considered the most efficient was to attach the lifting cables directly to the top chord near the ends of the bridge.
DEDICATION

This thesis is dedicated to my loving husband, Randy, and my beautiful daughter, Alexis Kate.
LIST OF ABBREVIATIONS AND SYMBOLS

*A* Area

*A_f* Area of compression flange

*A_g* Gross area

*A_t* Tributary area

*A_w* Area of the web

*B* Factor for bending stress of web-tapered members

*D* Dead load due to weight of bridge and decking system

*deg* Degrees

*d_o* Depth of smaller end of unbraced tapered segment

*E* Modulus of elasticity

*ft* Feet

*F_{by}* Flexural stress for tapered members

*F_{sy}* Stress for tapered members

*F_{wy}* Stress for tapered members

*F_y* Yield stress

*F_{yw}* Yield stress of web

*G* Specific gravity of southern pine

*h_o* Factor for web-tapered members

*h_w* Factor for web-tapered members
\( I \)  Moment of inertia

in  Inch

\( J \)  Torsional constant

\( K \)  Effective length factor

k  Kips

L  Live Load due to pedestrian load and truck load

\( L_u \)  Unbraced length of member

lb  Pound

m.c.  Moisture content of wood

min  Minute

\( M_n \)  Nominal flexural strength

\( M_u \)  Required flexural strength

pcf  Pounds per cubic feet

\( P_u \)  Required axial strength (tension or compression)

plf  Pounds per linear feet

\( P_n \)  Nominal axial strength (tension or compression)

psf  Pounds per square feet

psi  Pounds per square inch

\( Q \)  Reduction factor for slender compression elements

\( r \)  Radius of gyration about governing axis of buckling

\( r_{To} \)  Radius of gyration of the compression flange plus one third of the compression web area

\( S \)  Buckling ratio

\( S_x \)  Elastic section modulus about principal axis
sec  Second
$V_n$  Nominal shear strength
W  Wind load
$x$  Subscript relating symbol to member strong axis
$y$  Subscript relating symbol to member weak axis
$Z$  Plastic section modulus
$z$  Subscript relating symbol to member longitudinal axis
$\gamma$  Depth tapering ratio
$\lambda_c$  Column slenderness parameter
$\lambda_{eff}$  Effective slenderness ratio
$\phi_b$  Resistance factor for flexure
$\phi_c$  Resistance factor for compression
$\phi_t$  Resistance factor for tension
$\phi_v$  Resistance factor for shear
$\pi$  Pi
ACKNOWLEDGEMENTS

I would like to take this opportunity to recognize the people responsible for the completion of this thesis. First of all, I would like to thank Dr. James Richardson for his considerable patience and time spent guiding me during this process. His knowledge and experience proved to be invaluable to me. I would also like to thank Dr. Michael Triche and Dr. Garry Warren for serving on my thesis committee, and providing me with their knowledge and experience in their areas of study. My thanks is also extended to the Metallurgical and Materials Department in the College of Engineering for performing the metallographic analysis on the bridge samples, and also for allowing me the use of their lab equipment to perform the tensile tests on my bridge samples. I would like to acknowledge the Friends of Historic Downtown Northport for giving me the opportunity to work on this project.

I would like to extend my thanks to my loving parents for never failing to believe all that I could accomplish. I would especially like to thank my husband for his loving support and encouragement during my career as a graduate student. The completion of this thesis would not have been possible without him.
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CHAPTER 1

BACKGROUND AND LITERATURE REVIEW

The goal of this thesis was to plan the rehabilitation of a historic wrought iron bridge constructed in 1882. Specifically, the strength and ductility of the iron was determined through testing, a procedure for lifting the bridge in one piece from its current location was developed, and the foundations, approaches, and decking for the bridges at its new location was designed. This chapter discusses the background of the 1882 bridge and reviews literature on rehabilitation of similar bridges.

1.1 History of 1882 Bridge

The bridge studied for this thesis was one of three spans constructed across the Black Warrior River in Tuscaloosa, Alabama in 1882; hereafter, referred to as the 1882 Bridge. The bridge components were manufactured by Zenas King of King Iron and Bridge Company of Cleveland, Ohio, and is one of the oldest bridges in the southeast. It was replaced in 1897, and the three spans were relocated to cross streams and rivers throughout Tuscaloosa County. The section of bridge under investigation currently spans North River on an abandoned road near the Fayette/Tuscaloosa County line. The current rehabilitation plan for this bridge is to lift it off its supports using a crane, dismantle it for transportation, sand-blast and paint the bridge components, and re-assemble the bridge at
its final location, where it will serve as a pedestrian bridge. The new location of this bridge will be in Downtown Northport on the walking path atop the levee.

1.2  Review of Literature

The following literature review discusses the use of wrought iron as a structural material, evaluation of wrought iron properties, methods used to move historic bridges, and restoration of wrought iron members.

1.2.1 Wrought Iron as a Structural Material

Steel was introduced as a structural element in the mid 1880’s. Before that time, cast and wrought iron were used in structural applications. Wrought iron bridges were designed in the United States by Zenas King, Thomas W. H. Moseley, and Squire Whipple. Squire Whipple who was known as the “Father of Iron Bridges” patented and built the first successful iron bridge in the United States (Griggs, 2001). Thomas W. H. Moseley was given a patent in 1857 for a bowstring “truss” bridge which was fabricated entirely of wrought iron plate, bar, and strap stock (Griggs, 1997). The 1882 Bridge makes use of Zenas’ 1861 patent for the bowstring truss made with tubular sections (Pullaro 2001). Squire Whipple wrote, in his 1847 book on bridge building,

“Cast iron will resist a greater crushing force than any other substance, whose cost will admit of its being used as a building material. Steel has a greater power of resistance, but its cost precludes its use as a material for building. Wrought iron resists nearly equally with cast iron, but its cost is twice as great, which gives the cast iron entirely the advantage. On the other hand, wrought iron resists a tensile force nearly four times as well as cast iron…..It would seem then, that wrought iron for tension, and cast
iron for thrust, were the best materials that could be employed for building bridges.” (Griggs, 2001)

Another bridge engineer who appreciated this method was Francis Lowthorp. The Walnut Street Bridge in Pennsylvania makes use of wrought iron for the tension diagonals, bottom chord, and the sway bracing, and uses cast iron for the compression and bending members such as the top chord and floor beams (Green, Connor, and Higgins, 1999).

Early American iron makers had poor quality control over the iron’s ductility. As a result, many iron structures collapsed (Gordon and Knopf, 2005). Steel eventually replaced iron as a structural material based on its higher strength and better methods of uniform quality (Pullaro, 2001).

1.2.2 Evaluating Wrought Iron

Material properties can be evaluated in several ways including a metallographic analysis, material tests, and non-destructive tests. Metallographic analysis in conjunction with tensile testing is generally used to analyze bridges constructed during the change-over period of wrought-iron to steel which occurred from the mid 1880’s to 1900 (Pullaro, 2001). Wrought iron is a composite material consisting of a metal matrix and nonmetallic slag inclusions retained from the manufacturing process. Because it has properties distinctly different from the steel used in modern bridge construction, the methods and standards applicable to bridge steel are often inappropriate for the evaluation of these older materials (Gordon and Knopf, 2005).
1.2.2.1 Metallographic Analysis

A metallographic analysis can establish the microstructure of the material including the carbon and phosphorous content, the ferrite grain size, and amount and distribution of the included slag, and any inadequacies it may have (Gordon and Knopf, 2005).

The Walnut Street Bridge, which crosses the Tennessee River in Chattanooga, was rehabilitated in the mid-1980s after it was closed due to concerns regarding its structural integrity (Pullaro, 2001). Since there was uncertainty as to whether or not the bridge consisted of wrought iron members, a metallographic analysis was performed (Pullaro, 2001). The samples were taken from various bridge members in order to determine the grain structure of the material, which established that the metal was actually mild steel (Pullaro, 2001).

1.2.2.2 Material Testing

The capacity of wrought iron to serve in a bridge structure primarily depends on attaining a balance between two of its physical properties, strength and toughness; very strong iron may lack toughness, and is likely to fail by brittle fracture (Gordon and Knopf, 2005). Iron with good toughness will deform by plastic flow before ultimately failing by ductile rupture (Gordon and Knopf, 2005). Performing a material test on representative samples can reveal the strength and toughness of the bridge material.

In order to determine the quality of the wrought iron in the Keysville Road Bridge in Maryland, a tensile test was performed (Fu and Harwood, 2000). The samples were
taken from various members of the bridge, which established the yield stress of the material (Fu and Harwood, 2000).

When the Walnut Street Bridge in Pennsylvania was rehabilitated, it was necessary to evaluate the cast and wrought iron members of the bridge. To distinguish the mechanical behavior of the cast iron, material tests in compression, flexure, and tension were conducted. Even though the wrought iron materials were to be replaced, tensile tests were conducted, as part of the rehabilitation plan, to ascertain its tensile behavior and toughness. The results of the testing indicated that the cast and wrought iron possesses significant compressive strength and ductility and notch toughness (Green, Connor, and Higgins, 1999).

1.2.2.3 Non-Destructive Testing

Nondestructive testing techniques, such as ultrasonic and dye-penetrant, are used to locate flaws and fatigue cracks in members. Ultrasonic tests make use of high frequency sound waves to detect discontinuities in the metal, including cracks, slag inclusions, gas pockets, and laminations (Pullaro, 2001).

The Walnut Street Bridge in Tennessee used ultrasonic testing on its members. This test did not detect any fatigue cracking or any internal flaws (Pullaro, 2001).

The testing plan for the Keysville Road Bridge in Maryland included a dye-penetrant test; this test was performed on the bottom chord and various cast iron connection parts (Fu and Harwood, 2000). No cracks were found and it was determined that the connections were re-usable (Fu and Harwood, 2000).
1.2.3 Relocation

Rehabilitation of a historic bridge frequently involves relocation of the bridge. The bridge can be disassembled and moved in pieces or it can be moved as a whole. Several examples of bridge relocation are described below.

In 1965, the Santa Fe Railroad removed through-truss spans and replaced them with deck girder spans taken from another part of the bridge (Anon, 1965). To replace the truss spans with the girder spans, the contractor for this project skid the truss bridge laterally off its old piers onto a steel bolster; the ends of the bridge were lifted using a crane while wagons were positioned under each girder (Anon, 1965). A tractor was used to tow each span to its final location (Anon, 1965).

In 1931, P.G. Lang, Jr. reported that after the Baltimore & Ohio Chicago Terminal Railroad completed its relocation of the South Branch of the Chicago River, they raised the railroad tracks west of the river crossing due to the separation of railroad track grades at Stewart Avenue. It was decided that the bridge would be raised to its new height by utilizing 16 jacks; the bridge was lifted at four points, using four jacks per point. In order to prevent tilting of the span during the jacking process, rocker shoes were used and each jack was equipped with a rocker or tilting head (Lang, Jr., 1931).

In 1942, a Canadian steel corporation relocated one of their coal bridges by moving it along the ground to its new location (Tenant, 1942). The bridge was supported by a pier at one end and a shear leg at the other; both ends were jacked up enough to allow railroad trucks to move into position under them (Tenant, 1942). When the trucks reached their final destination, the bridge was jacked again, the trucks removed, and the structure brought to rest on the permanent rails (Tenant, 1942).
NNW, Inc. engineered the relocation of an iron truss bridge similar to the 1882 Bridge. The bridge was picked up by cables attached to the top chords near the ends; a compression strut was added as bracing at the lifting points between the top chords to keep them from moving in together (NNW, 2006).

1.2.4 Rehabilitation

In the restoration of a historic bridge such as the 1882 Bridge, it is important that the bridge retain its authenticity. If some members are beyond repair, then there is the option of fabricating a member using the same material or substituting parts with steel or iron elements. Typically, modern steel members of the same shapes are used to replace damaged members. Wrought iron can only be manufactured by the puddle process and is almost unobtainable (Nieuwmeijer and Arends, 2003).

If disassembling the bridge becomes necessary, there are several techniques to consider when working with iron. Vern Messler, who has restored numerous iron and steel truss bridges, makes use of the air-carbon arc process to remove rivets (Messler, 2006). This process severs or removes the rivet by melting it while doing little to no damage to the parent metal. For those members with section loss due to corrosion, Messler uses a welding technique called padding. This process builds up worn members so they can later be machined down to their original size. For welding wrought iron, a procedure was developed by Lon Yost, a senior application engineer at Lincoln Electric, in which he concludes that wrought iron can be successfully welded using the SMAW process and that E7018 provides sufficient tolerances of base metal inconsistencies (Messler, 2006). He also notes that the heat input of arc welding reduces the mechanical
properties but not enough to impact the performance of the members (Messler, 2006). For wrought iron, the welding is done parallel to the iron and slag fiber lines, after which a hand grinder is used to grind the surface smooth (Messler, 2006).

To rehabilitate the 1864 Moseley Bridge after it fell into the canal, Griggs used a significant amount of welding and steel as a replacement for the original wrought iron members (Griggs, 2001). Since wrought iron plates were not available in the required dimensions, a template using the existing iron was made to fabricate the steel to the correct shape and dimensions (Griggs, 1997).

The Tennessee Walnut Street Bridge incorporates post-tensioning cables. Grade 270 coated prestressing strands of 0.6 inch diameter were used to reduce the stresses on the trusses (Pullaro, 2001). Instead of replacing the deteriorated members, the cables were attached to the truss parallel to the tension members (Pullaro, 2001).

When the Aldrich Change Bridge in New York collapsed into the creek, all the cast iron verticals were either broken or among those removed from the creek; since these members were beyond repair, the only option was to cast new members from ductile cast iron (Griggs, 2001). Since the top chord suffered longitudinal cracks and broken ends, the members were welded using Cronacast 211 which provides good steel to iron connection (Griggs, 2001). To repair the wrought iron diagonals, new threaded wrought iron pieces were welded to the ends of the diagonals to restore them to their original load-carrying capacity (Griggs, 2001).

The Walnut Street Bridge in Pennsylvania was removed from vehicular service in 1970 due to structural and functional deficiencies; the rehabilitation plan for this bridge preserved the cast iron members and replaced the damaged wrought iron members with
modern A36 round steel bars (Green, Connor, and Higgins, 1999). Connections between castings were reinforced with steel pipe to ensure a ductile connection between the members (Green, Connor, and Higgins, 1999).

When the Keysville Road Bridge was closed due to deterioration of its various members, the rehabilitation plan included replacing and restoring various members (Fu and Harwood, 2000). Even though the diagonals were severely bent, they were cold straightened on a hydraulic press and put back in service; the other members that were too damaged to be restored were replaced with A36 steel (Fu and Harwood, 2000).

1.3 Plan of Study

The objective of this study was to plan the rehabilitation of a historic wrought iron bridge. The necessary steps taken for the work on this thesis include reviewing literature on the rehabilitation of similar bridges, conducting a metallurgical analysis and tensile test on bridge samples, analyzing the bridge’s resistance to dead, live, and wind loads, designing various lifting procedures for relocation, and designing the decking system, abutments, and approaches. The literature review and history of the 1882 Bridge are described in this chapter. Chapter two discusses the geometry of the bridge, its current condition, and the results of the metallographic analysis and tensile tests. Chapter three describes the structural analysis procedure to determine the bridge response to dead, live, and wind loads at its new location. Chapter four analyzes different lifting methods that were considered. Chapter five describes the design of the decking system, abutment, and approaches for its final location. Finally, chapter six presents the summary and conclusions of this thesis, and proposes future research.
CHAPTER 2
BRIDGE CONDITION

This chapter discusses the configuration and dimensions of the bridge, its current location and condition, the results of the tensile testing, and the results of the metallographic analysis.

2.1 Bridge Configuration and Dimensions

The shape of the 1882 Bridge is a bowstring. The section of bridge under investigation is 144 feet long, 20 feet tall at its apex, and 21 feet wide measured from the outside edge of the floor beams. There are eight panels, each of which measures 18 feet. Figure 2.1 shows a cross-sectional view of the bridge, Fig. 2.2 shows an elevation view of the bridge, and Fig. 2.3 shows a plan view of the bridge.
FIG. 2.1. Cross-sectional View of Bridge

FIG. 2.2. Elevation View of Bridge
FIG. 2.3. Plan View of Bridge

This bridge does not utilize standard American Institute of Steel Construction (AISC) sections. The members comprising this bridge consist of plates, channels, and angles that have been riveted together to form tubes and I-shapes. The top chord is a hollow rectangular tube measuring 10 in. x 8.65 in. A cross-section of the top chord can be seen in Fig. 2.4. It was fabricated by riveting flat plates to two channels.

FIG. 2.4. Cross-sectional View of Top Chord

The bottom chord consists of two 1 in. x 4 in. rectangular bars which are separated by a distance of 2 in., which can be seen in Fig. 2.5.
FIG. 2.5. Cross-sectional View of Bottom Chord

The geometry of the floor beam is a tapered I-beam, which is 12 in. deep at the ends and 24 in. deep at its middle. Figure 2.6 shows a cross-section of the floor beam, and Fig. 2.7 shows an elevation view of the floor beam.

FIG. 2.6. Cross-sectional View of Floor Beam
At each panel point, webbed tapered vertical members extend from the top chord to the bottom chord which can be seen in Fig. 2.1. At the fifth, sixth and seventh panel point, webbed bracing is used as lateral support. The diagonal cross-bracing of this bridge within the plane of the truss and extending laterally from the truss is 1 in. diameter rods.

2.2 Current Condition of Bridge

The photograph in Fig. 2.8 shows the original location of the bridge as it was in 1882 spanning the Black Warrior River.
The bridge currently spans North River off an old abandoned road in Samantha, Alabama. It is located in north Tuscaloosa County in the Berry SE and New Lexington Quadrangles at T. 17 S., R. 10 W. Section 18. Its coordinates are 33 deg 34 min 43 sec N, 87 deg 37 min 23 sec W, which can be seen in Fig. 2.9.

**FIG. 2.8.** Original Location of 1882 Bridge
FIG. 2.9. Current Location of 1882 Bridge

As seen in Fig. 2.10, the 1882 bridge has been neglected. The surrounding vegetation has overtaken the bridge, and the floor decking is non-existent.

FIG. 2.10. Current Condition of 1882 Bridge
The wrought iron members have very little corrosion. Several members have been damaged or are missing. One of the vertical members at the second panel point is missing and has been replaced with a cable which can be seen in Fig. 2.11.

![Fig. 2.11. Cable Serving as a Vertical at Second Panel Point](image)

Fig. 2.12 is a photograph that shows a hole in the top chord where the diagonal cross bracing should be.
2.3 Tensile Testing

Tensile tests were performed on samples taken from members of the bridge. Four cylindrical samples of diameter 0.5 in. were taken from the cross bracing which can be seen in Fig. 2.13. Two rectangular samples, which were 1 in. by 0.4 in., were removed with a cutting torch from the top flange of a floor beam as shown in Fig. 2.14.

FIG. 2.12. Damaged Top Chord

FIG. 2.13. Typical Round Specimen
These samples were tested in tension with a testing machine at a constant elongation rate of 0.1in/min. Figure 2.15 shows a typical stress-strain diagram for the cylindrical sample. The strains were calculated from the cross-head displacements and therefore do not represent strain in the specimen. This diagram shows a clearly defined upper and lower yield point. The upper yield point is approximately 42,700 psi, and the lower yield point is approximately 38,500 psi. The wrought iron exhibits strain hardening in the plastic region. The specimen failed suddenly without a “necking down phase” which is typically seen in tensile tests of steel specimens. The ultimate stress of this sample was approximately 54,000 psi.
A typical stress-strain diagram for the rectangular sample is shown in Fig. 2.16. This sample did not have a very well defined yield point as compared to the cylindrical sample. Its yield point was approximately 29000 psi, and its ultimate stress was approximately 41,000 psi.

![Stress-Strain Diagram for Rectangular Specimen](image)

**FIG. 2.16.** Stress-Strain Diagram for Rectangular Specimen

One of the earliest specifications for wrought iron, published in 1873, called for a tensile strength of 55,114 – 60,045 psi and a minimum of 25% reduction in area in a broken test bar (Gordon and Knopf, 2005). The percentage of reduction in area is calculated by dividing the original area by the difference of the original area and the area at failure. A graph of the tensile strength versus reduction in area is plotted for the cylindrical and rectangular samples in Fig. 2.17. The minimum required strength and ductility from the 1873 specification is also shown.
The above results show that the test results of the samples taken from the bridge fall short of the early ductility specifications, which indicate the presence of brittle wrought iron. The variable nature of the strength and ductility of wrought iron does not mean that it is an inferior metal for continued use in structures (Gordon and Knopf, 2005).

According to Gordon and Knopf, the adequacy of bridge metal for continued service is determined by comparing the calculated stresses of the bridge members to the published standards of the allowable stresses. The allowable tensile stress of wrought iron specified in 1840 was 13,924 psi, and the allowable tensile stress of 1909 was 11,168 psi. The strength of wrought iron is expected to exceed the permissible stresses by a factor of safety of typically 3.5 (Gordon and Knopf, 2006). Comparison of the 1882 Bridge member stresses to the published allowable stresses show that the strength of the

FIG. 2.17. Plot of Tensile Strength-Percent Reduction in Area for Rectangular and Circular Specimens
rectangular sample exceeds the allowable stress by approximately 3.5, while the strength of the cylindrical sample exceeds the allowable stress by approximately 4.4. Table 2.1 compares the published allowable tensile stresses to the tensile strengths of the 1882 Bridge samples.

**TABLE 2.1.** Comparison of Ultimate Tensile Stress of Rectangular and Circular Specimen to Specified Allowable Tensile Stress

<table>
<thead>
<tr>
<th>Allowable tensile stress</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1840 specification</td>
<td>13924 psi</td>
<td></td>
</tr>
<tr>
<td>1909 specification</td>
<td>11168 psi</td>
<td></td>
</tr>
<tr>
<td>Ultimate tensile stress</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rectangular sample</td>
<td>29000 psi</td>
<td></td>
</tr>
<tr>
<td>Cylindrical sample</td>
<td>54000 psi</td>
<td></td>
</tr>
</tbody>
</table>

Since an extensometer was not used to measure specimen elongation, an accepted value of the modulus of elasticity for wrought iron was used, which was 28,500 ksi. Since the samples gave two different yield points that were not comparable, the lower value of 29000 psi was used in the structural analysis procedures.

2.4 Metallographic Analysis

A metallographic analysis was conducted on the rectangular and cylindrical specimens in order to determine its microstructure. A scanning electron microscope (SEM) was able to determine the slag distribution of the material, and the elemental distribution was analyzed using an energy dispersive spectroscopy (EDS).
2.4.1 Analysis of Cylindrical Sample

Figure 2.18 shows the slag inclusions of the cylindrical sample, which are indicated by the arrows, as seen at a magnification of 125. The slag inclusions do not lie in the same direction, which is not typical of wrought iron because slag inclusions are normally elongated along the worked direction (Henry, 2006). A magnified view of the slag inclusion at Point “A” is shown in Fig. 2.19. Besides the larger inclusion, smaller inclusions can be seen along with some porosity.

FIG. 2.18. SEM Image of Circular Specimen
FIG. 2.19. SEM Image of Slag Inclusion

Table 2.2 shows the elemental analysis at Point “B” of Fig. 2.19. The table below shows the elemental analysis for “C” as referenced in Fig. 2.19. Comparison of the elemental analysis for “B” and “C” shows that there are many different elements in “B” at the slag inclusion than there are for “C” which is not a slag inclusion.

<table>
<thead>
<tr>
<th>Element</th>
<th>Weight %</th>
<th>Atomic %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oxygen</td>
<td>27.2</td>
<td>52.28</td>
</tr>
<tr>
<td>Magnesium</td>
<td>0.36</td>
<td>0.45</td>
</tr>
<tr>
<td>Aluminum</td>
<td>0.23</td>
<td>0.26</td>
</tr>
<tr>
<td>Silicon</td>
<td>3.88</td>
<td>4.25</td>
</tr>
<tr>
<td>Phosphorous</td>
<td>8.44</td>
<td>8.38</td>
</tr>
<tr>
<td>Sulfur</td>
<td>0.53</td>
<td>0.51</td>
</tr>
<tr>
<td>Calcium</td>
<td>5.42</td>
<td>4.16</td>
</tr>
<tr>
<td>Manganese</td>
<td>0.99</td>
<td>0.56</td>
</tr>
<tr>
<td>Iron</td>
<td>52.95</td>
<td>29.16</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>100.00</strong></td>
<td><strong>100.01</strong></td>
</tr>
</tbody>
</table>
TABLE 2.3. EDS Results from Point "C" from Fig. 2.19

<table>
<thead>
<tr>
<th>Element</th>
<th>Weight %</th>
<th>Atomic %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oxygen</td>
<td>2.32</td>
<td>7.67</td>
</tr>
<tr>
<td>Iron</td>
<td>97.68</td>
<td>92.33</td>
</tr>
<tr>
<td>Total</td>
<td>100.00</td>
<td>100.00</td>
</tr>
</tbody>
</table>

2.4.2 Analysis of Rectangular Bar Sample

Figure 2.20 shows an SEM image of the rectangular specimen at a magnification of 250. The slag inclusions are indicated by the arrows, and are elongated along the same direction, unlike the slag inclusions of the cylindrical sample.

FIG. 2.20. SEM Image of Rectangular Specimen
Smaller slag inclusions were observed which can be seen in Fig. 2.21. Besides the larger slag inclusion, small slag inclusions can be seen along with some porosity.

![SEM Image of Slag Inclusion](image)

**FIG. 2.21.** SEM Image of Slag Inclusion

Table 2.4 shows the elemental analysis for Point “D” as referenced in Fig. 2.21, and Table 2.5 shows the elemental analysis for Point “E” as referenced in Fig. 2.21.

**TABLE 2.4.** EDS Results from Point "D" from Fig. 2.21

<table>
<thead>
<tr>
<th>Element</th>
<th>Weight %</th>
<th>Atomic %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oxygen</td>
<td>28.33</td>
<td>54.20</td>
</tr>
<tr>
<td>Aluminum</td>
<td>0.66</td>
<td>0.75</td>
</tr>
<tr>
<td>Silicon</td>
<td>9.40</td>
<td>10.24</td>
</tr>
<tr>
<td>Phosphorous</td>
<td>2.20</td>
<td>2.17</td>
</tr>
<tr>
<td>Calcium</td>
<td>0.36</td>
<td>0.28</td>
</tr>
<tr>
<td>Manganese</td>
<td>0.87</td>
<td>0.48</td>
</tr>
<tr>
<td>Iron</td>
<td>58.19</td>
<td>31.89</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td><strong>100.01</strong></td>
<td><strong>100.01</strong></td>
</tr>
</tbody>
</table>
TABLE 2.5. EDS Results from Point "E" from Fig. 2.21

<table>
<thead>
<tr>
<th>Element</th>
<th>Weight %</th>
<th>Atomic %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oxygen</td>
<td>2.26</td>
<td>7.48</td>
</tr>
<tr>
<td>Iron</td>
<td>97.74</td>
<td>92.52</td>
</tr>
<tr>
<td>TOTAL</td>
<td>100.00</td>
<td>100.00</td>
</tr>
</tbody>
</table>

2.4.1 Results of Metallographic Analysis

The results of the metallographic analysis for the rectangular and cylindrical sample indicate that the structures are typical of wrought iron (Henry, 2006). There is a noteworthy difference in the elemental analysis of the slag inclusions compared to the base material (iron). The EDS results of the slag inclusions of the two samples indicate the presence of numerous elements, in addition to iron, while the EDS results taken from other points along the samples show that the only elements that were detectable were oxygen and iron.

Chapter one discussed the fact that carbon content can influence the strength and ductility of wrought iron. Wrought iron typically has less than 0.10% carbon (Henry, 2006). Since the detectability limit of the EDS system used is less than 0.1%, the elemental analysis of the samples show that there is not a significant amount of carbon in the samples. Phosphorous can also have as great an impact on the strength and ductility of wrought iron as carbon (Gordon and Knopf, 2006). The elemental analysis of the slag inclusions on both samples indicates an existence of phosphorous. The phosphorous content of the cylindrical sample is 8.44%, and the phosphorous content of the rectangular sample is 2.20%. Research by Gordon and Knopf concluded that phosphorous can reduce the ductility of wrought iron, and can also give inadequate toughness in load-bearing structures if the average content exceeds 0.3%. The elemental
analysis of the samples’ slag inclusion also indicates manganese. However, the manganese content in both samples is not a significant amount since the content for both is less than 1%. Manganese is another solute commonly present in wrought iron, but it does not have as much influence on the strength and ductility of the metal as phosphorous (Gordon and Knopf, 2006). Silicon was also detected in the samples’ slag inclusion. The cylindrical sample show a silicon content of 3.44%, and the rectangular sample show a silicon content of 9.4%. Even though silicon is a hardener, it is not a source of embrittlement (Gordon and Knopf, 2006).
CHAPTER 3

STRUCTURAL ANALYSIS

Chapter three presents a detailed discussion of the structural analysis of the 1882 Bridge. The analysis of the bridge is for its final location where it will serve as a pedestrian bridge.

The bridge was analyzed using RISA-3D, which is a commercial software program for structural analysis. Because the cross-sectional shapes of the members are not standard AISC shapes, RISA was not able to perform a full analysis of the bridge such as determining the various member strengths and calculating unity checks for the interaction equations of the members. A unity check is the ratio of the required strength over the strength capacity. A unity check of less than one means that the member forces do not exceed the member’s capacity, and a unity check of greater than one means that the forces exceed the member’s capacity. The member strengths and unity checks were calculated using an Excel spreadsheet and the steel design equations from the AISC Load and Resistance Factor Design (LRFD) Third Edition.

The expected loads on the bridge for its final location were determined by the AASHTO Bridge Design Guide. The expected loads include the weight of the structure, the weight of the wood decking system, a pedestrian live load, the moving load of the maintenance vehicle, and a wind load. The load combinations were governed by AISC LRFD Third Edition.
As a check on the above-mentioned model of the bridge (which will be designated the “structural model”, hereafter), a second approximate model was made in RISA-3D using equivalent steel shapes. The approximate model incorporated AISC standard shapes, with section properties similar to the actual section properties. RISA was able to perform a full analysis of the approximate model of the bridge. The equivalent steel model is sufficient for an approximate, conservative, and quick analysis, but it lacks the accuracy of the structural model. The equivalent steel model was compared to the structural model as a way to check the reasonableness of the structural model.

The section to follow gives a detailed discussion of the calculation of the structural model that was created in RISA and the calculation of the expected loads on the bridge. Next, the calculation of the member strengths will be discussed which include the design tensile strength, the design compressive strength, and the design flexural strength of the top chord, bottom chord, floor beams, cross-bracing, and top lateral bracing. Then, an approximate model using equivalent steel shapes will be discussed. Finally, a comparison will be given between the structural model and the approximate model.

3.1 Structural Model

A 3D finite element model of the 1882 Bridge was created in RISA-3D. The model can be seen in Fig. 3-1, which shows the orientation to the global axes.
The planes of the trusses are parallel with the x-y plane. The floor beams are parallel with the y-z plane. The supports of the bridge were modeled as pins and rollers. The arched top chord of the bridge was modeled with 75 straight members. The ends of each top chord member were modeled as having fixed ends, meaning bending moments were transferred through the nodes between the members. The bottom of each vertical was also modeled as having fixed ends. The ends of all other members of the bridge were modeled as pinned ends, meaning bending moments were not transferred between the members. Fig. 3-2 shows the boundary conditions and end releases for one truss, where the circles represent pinned ends.
Since the 1882 Bridge does not utilize standard AISC shapes, which are in the shape files in RISA, non-standard shapes were used. In the model, the diagonal cross-bracing were analyzed as tension only members.

3.1.1 Loading

Once the model was created, the loading conditions were analyzed. The loads on the bridge include the dead load, live load, and wind load. The dead load includes the weight of the bridge and the weight of the decking system. The live loads include the pedestrian load and a moving truck load.

3.1.1.1 Dead Loads

The dead and live loads were entered as distributed loads along each floor beam, which can be seen in Fig. 3-3.
FIG. 3-3. Distribution of Dead and Live Loads

The dead loads applied to the floor beams represent the weight of the wood decking system. The wood deck and stringer design is presented in Section 5.1. The density of the wood decking system was calculated using Equation 3-1 as specified by the 2005 Edition of the National Design Specification (NDS) Supplement.

\[
\text{Density} = 62.4 \left[ \frac{G}{1 + G(0.009)(m.c.)} \right] \left[ 1 + \frac{m.c.}{100} \right] \tag{3-1}
\]

The variable, \(G\), represents the specific gravity of southern pine, and m.c. is the moisture content of wood which is 19%. The calculated density was multiplied by the volume of the wood to obtain a concentrated dead load. The “road” area of the bridge was divided by the concentrated dead load to obtain a surface load. The surface load was multiplied by the tributary width of the floor beams to obtain a distributed load along each floor beam.
The dead load represented by the self weight of each member was applied “automatically” by RISA as a distributed load along the member. The total weight of the bridge was determined to be approximately 38,000 lbs.

3.1.1.2 Live Loads

The live load includes the pedestrian load and the moving truck load. The pedestrian load equaled 85 pounds per square feet (psf) as specified by AASHTO. Using a tributary width for each floor beam, the loading on the interior floor beams were calculated as having a distributed load of 1,530 pounds per linear feet (plf). The loads on the exterior floor beams were calculated to be 765 plf.

The truck load was assumed to be a Ford F-150 with towing capacity, which was approximately 10,000 lbs. The specifications for the truck were attained from the Ford Truck website. The specifications needed to calculate the moving truck load include the distance between the front and rear axles which is 126 in. and also the distance between the two front tires which is 67 in. The moving truck load was analyzed as two wheel line loads moving the length of the bridge, which can be seen in Fig. 3-4.

FIG. 3-4. Wheel Lines for Moving Load
The rear axle was assumed to take 60% of the vehicle weight, and the front axle was assumed to take 40% of the vehicle weight. The total weight of the front axle was calculated to be 4000 lbs, and the total weight of the rear axle was calculated to be 6000 lbs. Figure 3-5 shows the deflected shape of the bridge when subjected to dead and live load.

FIG. 3-5. Deflected Shape of Bridge Under Dead and Live Loads
3.1.1.3 Wind Loads

The wind load was calculated based on AASHTO’s design guide for highway bridges. The wind load was applied horizontally at right angles to the longitudinal axis of the bridge along the top and bottom chords as seen in Fig. 3-6.

![Distribution of Wind Loads Applied to Top and Bottom Chord](image)

**FIG. 3-6.** Distribution of Wind Loads Applied to Top and Bottom Chord

The intensity of the wind load was 75 psf for a base wind velocity of 100 miles per hour. The exposed areas of the members, deck, and railing were calculated to get the total wind load. Half of the load was applied to the top chord and the other half was applied to the bottom chord. The distributed load on the top chord was calculated to be 72 plf, and the distributed load applied to the bottom chord was calculated to be 93 plf. Figure 3-7 shows the deflected shape of the bridge when subjected to wind loads.
3.2 Member Forces

The member forces that were analyzed include bending, shear, and axial forces. The forces that will be presented are the maximum forces taken from each type of member which include the top chord, bottom chord, floor beam, verticals, and diagonals. The following load combinations, which came from AISC LRFD Third Edition, were considered:

1. $1.2D + 1.6L$ (pedestrian load);
2. $1.2D + 1.6W + 0.5L$ (pedestrian load);
3. $1.2D + 1.6L$ (moving truck load); and
4. $1.2D + 1.6W + 0.5L$ (moving truck load).

D represents dead load, L represents live load, and W represents wind load. Since the software program used was not able to analyze non-standard shapes, the member forces from each load combination were entered into an Excel spreadsheet to calculate the unity check of each member. The factored forces for each load combination are shown in
Tables 3-1 through 3-4. The positive forces in the tables are compression forces, and the negative forces are tensile forces.

**TABLE 3-1.** Forces for LC I: 1.2D+1.6L(Ped)

<table>
<thead>
<tr>
<th></th>
<th>Axial, k</th>
<th>y Shear, k</th>
<th>z Shear, k</th>
<th>y-y Moment, k-ft</th>
<th>z-z Moment, k-ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top chord</td>
<td>197.4</td>
<td>12.1</td>
<td>0.0</td>
<td>0.0</td>
<td>41.5</td>
</tr>
<tr>
<td>Bottom chord</td>
<td>-163.1</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Floor beam</td>
<td>6.4</td>
<td>25.9</td>
<td>0.0</td>
<td>0.0</td>
<td>123.0</td>
</tr>
<tr>
<td>Diagonal</td>
<td>-11.9</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Vertical</td>
<td>-17.9</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

**TABLE 3-2.** Forces for LC II: 1.2D+1.6W+0.5L(Ped)

<table>
<thead>
<tr>
<th></th>
<th>Axial, k</th>
<th>y Shear, k</th>
<th>z Shear, k</th>
<th>y-y Moment, k-ft</th>
<th>z-z Moment, k-ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top chord</td>
<td>88.2</td>
<td>5.6</td>
<td>3.3</td>
<td>29.9</td>
<td>19.9</td>
</tr>
<tr>
<td>Bottom chord</td>
<td>-27.1</td>
<td>0.0</td>
<td>1.3</td>
<td>5.9</td>
<td>0.0</td>
</tr>
<tr>
<td>Floor beam</td>
<td>17.6</td>
<td>9.9</td>
<td>0.0</td>
<td>0.0</td>
<td>47.1</td>
</tr>
<tr>
<td>Diagonal</td>
<td>-24.1</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Vertical</td>
<td>-3.7</td>
<td>18.1</td>
<td>0.0</td>
<td>0.0</td>
<td>27.2</td>
</tr>
</tbody>
</table>

**TABLE 3-3.** Forces for LC III: 1.2D+1.6L(Truck)

<table>
<thead>
<tr>
<th></th>
<th>Axial, k</th>
<th>y Shear, k</th>
<th>z Shear, k</th>
<th>y-y Moment, k-ft</th>
<th>z-z Moment, k-ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top chord</td>
<td>36.9</td>
<td>2.5</td>
<td>0.0</td>
<td>0.0</td>
<td>15.6</td>
</tr>
<tr>
<td>Bottom chord</td>
<td>-31.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Floor beam</td>
<td>0.0</td>
<td>10.5</td>
<td>0.0</td>
<td>0.0</td>
<td>56.6</td>
</tr>
<tr>
<td>Diagonal</td>
<td>-7.4</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Vertical</td>
<td>0.5</td>
<td>1.8</td>
<td>0.0</td>
<td>0.0</td>
<td>15.6</td>
</tr>
</tbody>
</table>
3.3 Calculating Member Strengths

The physical properties of the members were calculated in a spreadsheet and entered into RISA’s shape database. The properties that were calculated in Excel include the areas, moments of inertia about the strong and weak axes, and the torsional constants. Table 3-5 shows the cross-sectional properties that were calculated for each member.

### TABLE 3-5. Cross-Sectional Properties of Bridge Members

<table>
<thead>
<tr>
<th>Member</th>
<th>$A, \text{in}^2$</th>
<th>$I_{zz}, \text{in}^4$</th>
<th>$I_{yy}, \text{in}^4$</th>
<th>$J, \text{in}^4$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom Chord</td>
<td>8</td>
<td>10.67</td>
<td>2.667</td>
<td>7.324</td>
</tr>
<tr>
<td>Top Chord</td>
<td>11.7</td>
<td>175.9</td>
<td>140.4</td>
<td>238.6</td>
</tr>
<tr>
<td>Floor beam, small end</td>
<td>8.7</td>
<td>219.7</td>
<td>18.55</td>
<td>0.4613</td>
</tr>
<tr>
<td>Floor beam, large end</td>
<td>12</td>
<td>1065.6</td>
<td>18.571</td>
<td>0.5445</td>
</tr>
</tbody>
</table>

Figures 2-4 through 2-7 in chapter two show the cross-sections of the bridge members, however, each cross-section is shown again in a separate figure in this chapter. RISA cannot perform a unity check on non-standard shapes, because it does not know which specification in the AISC manual the member falls under. RISA would need to know, among other things, whether or not the member was doubly or singly symmetric. Since there is not a way to specify this information for non-standard shapes in RISA, the various strengths of the members were calculated in an Excel spreadsheet using the appropriate equations from the AISC manual.
3.3.1 Top Chord

The top chord can be described as a doubly symmetric built up box section. It is built up with channels and rectangular plates. The rectangular plates are riveted to the sides of two channels. Fig. 3-8 shows a cross-sectional view of the top chord.

\[ \lambda \phi = \frac{F_y}{(0.658 \lambda_{cr}^2) F_y} \]  

\[ (3-2) \]

The column slenderness parameter is given by Equation 3-3.

Since the top chord is a doubly symmetric member, it was investigated for the limit states of flexural buckling and torsional buckling. Based on the calculations, the controlling failure mode is flexural buckling. Equation 3-2 was used to determine its design compressive strength.

The column slenderness parameter is given by Equation 3-3.
The top chord was designed for flexure. Because the top chord is compact and considered to be laterally braced, the only limit state investigated was yielding. Equation 3-4 was used to determine its design flexural strength.

\[ \phi_b M_n = 0.9 F_y Z \]  

(3-4)

The tensile capacity of the top chord was determined by the limit states of yielding in the gross section. Fracture in the net section was not considered. Equation 3-5 was used to determine the design tensile strength of the top chord.

\[ \phi_t P_n = 0.9 F_y A_g \]  

(3-5)

The design shear strength of the top chord was calculated using Equation 3-6.

\[ \phi_i V_n = 0.9(0.6) F_{yw} A_w \]  

(3-6)

3.3.2 Bottom Chord

The bottom chord can be described as a compact, doubly symmetric section, which can be seen in Fig. 3-9.
Equation 3-9 was used to calculate the design tensile strength for the bottom chord. The limit state investigated was yielding of the gross section.

\[ \phi_n P_n = 0.9 F_y A_g \]  

(3-9)

The bottom chord was subject to bending about its minor axis when the load combinations including the wind load was applied. Based on the unbraced length of the bottom chord and the corresponding limiting length parameters, the bottom chord is suspect to yielding and lateral torsional buckling. Since lateral torsional buckling is not applicable to members subject to bending about the minor axis, the limit state of yielding controls if the shape is compact. Equation 3-10 was used to calculate the design flexural strength of the bottom chord:

\[ \phi_b M_n = \phi_b F_y Z_y \]  

(3-10)

Even though the bottom chord is a tension member under gravity loads, it is placed in compression during the lifting methods which will be discussed in the following chapter. The bottom chord was investigated for flexural buckling and torsional...
buckling. The controlling failure mode for this compact shape is flexural buckling.

Equation 3-11 was used to determine its design compressive strength:

\[ \phi P_n = 0.85 A_y (0.658 \frac{t}{\lambda})^2 F_y \]  

(3-11)

3.3.3 Floor Beam

The floor beam can be described as a singly symmetric, compact, built-up, web-tapered member. It is built up with angles, a rectangular plate, and is symmetric about its y-axis. The angles are riveted to the rectangular plate in order to resemble an "I" shape. Fig. 3-10 below shows a cross-sectional view of the floor beam at its midpoint and Fig. 3-11 shows an elevation view of the tapered floor beam.

**FIG. 3-10.** Cross-Sectional View of Floor Beam
Equation 3-12 was used to calculate its tensile strength, where the gross area is conservatively taken at the small end of the floor beam:

$$\phi P_n = 0.9 F_y A_g$$  \hspace{1cm} (3-12)

Equation 3-13 was used to calculate the compressive strength of the floor beam. This equation applies to web tapered members, with a linearly varying depth, flanges with equal and constant area, and one axis of symmetry perpendicular to the plane of bending. The floor beam was investigated for the limit states of flexural buckling and flexural-torsional buckling. The controlling failure mode is flexural-torsional buckling. The design compressive strength of the tapered floor beam was calculated in the same manner as a prismatic member with the following modification:

$$\phi_c P_n = 0.85 A_g (0.658^{\lambda_{eff}^2}) F_y$$  \hspace{1cm} (3-13)

The effective slenderness parameter, $\lambda_{eff}$, is the effective slenderness parameter which is defined by Equation 3-14, and the column slenderness parameter, $S$, is defined by Equation 3-15.

$$\dot{\lambda}_{eff} = \frac{S}{\pi} \sqrt{\frac{Q F_y}{E}}$$  \hspace{1cm} (3-14)
The unbraced length of the floor beam is taken as 18 feet. The reduction factor, \( Q \), is taken as one, because the floor beam is not considered a slender element.

The AISC manual specifies that the design flexural strength be determined for the properties of the critical section under consideration of the floor beam. The floor beam was investigated for the limit state of lateral torsional buckling. Since the moment diagram for the floor beam is parabolic, the flexural strength of the floor beam was calculated for the small end, the quarter point, and the midpoint. The design flexural strength was calculated using Equation 3-16.

\[
\phi_b M_n = 0.9(5/3)S'_x F_{by}. 
\]  
(3-16)

The flexural stress is computed according to equation 3-17,

\[
F_{by} = \frac{2}{3} \left[ 1.0 - \frac{F_y}{6B \sqrt{F_{sx}^2 + F_{swy}^2}} \right] F_y, 
\]  
(3-17)

and \( F_{sx} \) and \( F_{swy} \) are defined by Equations 3-18 and 3-19, respectively.

\[
F_{sx} = \frac{0.41E}{h_s L d_o / A_f} 
\]  
(3-18)

\[
F_{swy} = \frac{5.9E}{(h_w L / r_{to})^2} 
\]  
(3-19)

In Equations 3-20 through 3-23, \( \gamma \) is a depth tapering ratio, \( h_s \) and \( h_w \) are factors for web-tapered members, and \( B \) is a bending stress factor for web-tapered members.

\[
\gamma = \frac{d_L - d_o}{d_o} 
\]  
(3-20)

\[
h_s = 1.0 + 0.023 \gamma \sqrt{L d_o / A_f} 
\]  
(3-21)
\[ h_w = 1.0 + 0.00385\gamma\sqrt{L/r_{lo}} \]  \hspace{1cm} (3-22)

\[ B = \frac{1.75}{1.0 + 0.25\sqrt{1.0}} = 1.4 \]  \hspace{1cm} (3-23)

At the small end of the floor beam, the design flexural strength was calculated to be 56 k-ft. At the quarter point, it was calculated to be 97 k-ft, and at the midpoint, it was calculated to be 142 k-ft.

The shear strength of the floor beam was calculated using Equation 3-24.

\[ \phi V_n = 0.9(0.6)F_{yw}A_w \]  \hspace{1cm} (3-24)

3.3.4 Verticals

The verticals can be described as a singly symmetric, web-tapered member. The chords of the verticals resemble a cruciform section, and the web of the verticals is a bar shape. In Fig. 3-12, Fig. 3-13(a), and Fig. 3-13(b), the elevation view and the cross-section view of the vertical members can be seen.

\[ \text{FIG. 3-12. Elevation View of Vertical} \]
Because the verticals at every panel points had tapered webbing, an average area was calculated to determine its tensile strength. Equation 3-25 was used to calculate its tensile strength:

\[ \phi_k P_n = 0.9 F_y A_k \]  (3-25)

The design flexural strength of the verticals was calculated based on the limit state of yielding using Equation 3-26.

\[ \phi_b M_n = 0.9 Z_x F_y \]  (3-26)

The plastic section modulus, \( Z_x \), was computed at the midpoint of the vertical.

The shear strength of the vertical was calculated using Equation 3-27.

\[ \phi_s V_n = 0.9(0.6) F_y A_w \]  (3-27)

The area of the web was calculated at the midpoint of the vertical.
3.3.5 Top Lateral Bracing

The top lateral cross bracing at the third and seventh panel could be described as a truss with the top and bottom chords being a double angle and the web being 0.75 in. diameter cross bracing. Fig. 3-14 and Fig. 3-15 show an elevation view and a cross-sectional view of the bracing. Assumptions regarding this member’s dimensions and spacing had to be made since access to this portion of the bridge was difficult. Given that the top and bottom chord of the bracing at this location was analyzed as a double angle, RISA-3D was able to calculate the strengths of this member. The design compressive strength of the top double angle was calculated to be approximately 1.8 kips. Its design flexural strength was approximately 3.6 kip-ft, and its design tensile strength was 61 kips.

**FIG. 3-14.** Elevation View of Top Lateral Bracing at Third and Seventh Panel
FIG. 3-15. Cross-Sectional View of Top Lateral Bracing at the Third and Seventh Panel

The top lateral cross bracing at the fourth and sixth panel could be described as an I-shape with a depth of four inches and a flange width of two inches. Fig. 3-16 shows a cross-sectional view of the top lateral bracing.

FIG. 3-16. Cross-Sectional View of the Top Lateral Bracing at the Fourth and Sixth Panels
The top lateral cross bracing at the fifth panel could be described as a truss with the top and bottom chords resembling a I shape with a depth of four inches and a flange width of two inches and the web being a 0.75 in. diameter cross bracing rod, which can be seen in Fig. 3-17 and Fig. 3-18.

FIG. 3-17. Elevation View of the Top Lateral Bracing at the Fifth Panel

FIG. 3-18. Cross-Sectional View of the Top Lateral Bracing at the Fifth Panel
Assumptions were made regarding the dimensions of the top and bottom chords at the fourth, fifth, and sixth panel points since access to this portion of the bridge was difficult. Because it was analyzed as an I-shape, RISA-3D was able to calculate the strengths of these members. Its compressive strength was approximately 0.303 kips. Its flexural strength was approximately 0.364 kip-ft, and its tensile strength was approximately 22.9 kips.

3.3.6 Diagonal Cross Bracing

The diagonal cross bracing for the trusses, floor beams, and top lateral bracing is 0.75 in. round bars.

The compressive strength and flexural strength of the cross-bracing was neglected because it was analyzed as a tension only member. The tensile strength of the diagonal cross-bracing was calculated according to Equation 3-28:

$$\phi P_a = 0.9 F_y A_g$$  \hspace{1cm} (3-28)

A summary of the design strengths that was calculated in Excel for the top chord, bottom chord, floor beam, cross-bracing, and verticals can be seen in Tables 3-6 through 3-9.

**TABLE 3-6. Design Tensile Strengths**

<table>
<thead>
<tr>
<th>Member</th>
<th>Design tensile strength, k</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top chord</td>
<td>305.4</td>
</tr>
<tr>
<td>Bottom chord</td>
<td>209</td>
</tr>
<tr>
<td>Floor beam</td>
<td>227</td>
</tr>
<tr>
<td>Cross-bracing</td>
<td>11.5</td>
</tr>
<tr>
<td>Verticals</td>
<td>59.4</td>
</tr>
</tbody>
</table>
TABLE 3-7. Design Compressive Strengths

<table>
<thead>
<tr>
<th>Member</th>
<th>Controlling Failure Mode</th>
<th>Design Compressive Strength, k</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top Chord</td>
<td>Flexural Buckling</td>
<td>244</td>
</tr>
<tr>
<td>Bottom Chord</td>
<td>Flexural Buckling</td>
<td>2</td>
</tr>
<tr>
<td>Floor Beam</td>
<td>Flexural-Torsional Buckling</td>
<td>83</td>
</tr>
</tbody>
</table>

TABLE 3-8. Design Flexural Strengths

<table>
<thead>
<tr>
<th>Member</th>
<th>Controlling failure mode</th>
<th>Design flexural strength, k-ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top chord</td>
<td>Yielding</td>
<td>98</td>
</tr>
<tr>
<td>Bottom chord</td>
<td>Lateral torsional buckling</td>
<td>17.4</td>
</tr>
<tr>
<td>Floor beam</td>
<td>Lateral torsional buckling</td>
<td>98, at quarter point</td>
</tr>
<tr>
<td>Vertical</td>
<td>Lateral torsional buckling</td>
<td>105</td>
</tr>
</tbody>
</table>

TABLE 3-9. Design Shear Strengths

<table>
<thead>
<tr>
<th>Member</th>
<th>Design shear strength, k</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top chord</td>
<td>101.8</td>
</tr>
<tr>
<td>Floor beam</td>
<td>51.6</td>
</tr>
<tr>
<td>Verticals</td>
<td>28.2</td>
</tr>
</tbody>
</table>

3.4 Results of Structural Model Analysis

The unity checks for bridge members subject to a combination of bending and axial forces were calculated using Equation 3-29 and Equation 3-30 as shown below.

For \( \frac{P_u}{\phi P_n} \geq 0.2 \),

\[
\text{U.C.} = \frac{P_u}{\phi P_n} + \frac{8}{9} \left( \frac{M_{ux}}{\phi_b M_{ux}} + \frac{M_{uy}}{\phi_b M_{uy}} \right) \leq 1.0 \quad (3-29)
\]

For \( \frac{P_u}{\phi P_n} \angle 0.2 \),

\[
\text{U.C.} = \frac{P_u}{2\phi P_n} + \frac{M_{ux}}{\phi_b M_{ux}} + \frac{M_{uy}}{\phi_b M_{uy}} \leq 1.0 \quad (3-30)
\]
For members that were not subject to combined axial and bending forces, the ratio
of the factored load to the design strength was calculated. The sequence below shows a
typical calculation for computing the unity check of the top chord under LC II, using
Equation 3-30. The forces can be seen in Table 3-2, and the strength capacities can be
seen in Tables 3-7 and 3-8.

\[
\frac{88.25}{244} = 0.36 \leq 0.2, \quad \frac{88.25}{244} + \frac{8 \left( \frac{19.9}{98} + \frac{29.86}{100} \right)}{9} = 0.807 \leq 1.0
\]

The analysis of the first load combination, 1.2D + 1.6L(Pedestrian), showed that
the top chord, floor beam, and a couple of the diagonals were overstressed. The most
stressed member was the floor beam, with a unity check of approximately 1.29. Table 3-
10 shows a summary of the maximum unity check for each main member of the bridge.

**TABLE 3-10.** Unity Checks for LC I: 1.2D+1.6L (Ped)

<table>
<thead>
<tr>
<th>Member</th>
<th>Unity check</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top chord</td>
<td>1.16</td>
</tr>
<tr>
<td>Bottom chord</td>
<td>0.841</td>
</tr>
<tr>
<td>Floor beam</td>
<td>1.29</td>
</tr>
<tr>
<td>Vertical</td>
<td>0.398</td>
</tr>
<tr>
<td>Diagonal cross braces</td>
<td>1.03</td>
</tr>
</tbody>
</table>

The analysis of the second load combination, 1.2D+1.6W+0.5L(Pedestrian),
showed that the most stressed member was the diagonal with a unity check of
approximately 2.1. Table 3-11 shows a summary of the maximum unity check for each
main member of the bridge.
TABLE 3-11. Unity Checks for LC II: 1.2D+1.6W+0.5L (Ped)

<table>
<thead>
<tr>
<th>Member</th>
<th>Unity check</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top chord</td>
<td>0.807</td>
</tr>
<tr>
<td>Floor beam</td>
<td>0.639</td>
</tr>
<tr>
<td>Bottom chord</td>
<td>0.492</td>
</tr>
<tr>
<td>Verticals</td>
<td>0.167</td>
</tr>
<tr>
<td>Diagonal cross bracing</td>
<td>2.1</td>
</tr>
</tbody>
</table>

The third load combination, 1.2D+1.6L(Truck), yielded a unity check for the diagonal of approximately 0.642. Table 3-12 shows a summary of the unity checks for each main bridge member.

TABLE 3-12. Unity Checks for LC III: 1.2D+1.6L (Truck)

<table>
<thead>
<tr>
<th>Member</th>
<th>Unity check</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top chord</td>
<td>0.235</td>
</tr>
<tr>
<td>Bottom chord</td>
<td>0.148</td>
</tr>
<tr>
<td>Floor beam</td>
<td>0.578</td>
</tr>
<tr>
<td>Vertical</td>
<td>0.258</td>
</tr>
<tr>
<td>Diagonal</td>
<td>0.642</td>
</tr>
</tbody>
</table>

The results of the fourth combination, 1.2D+1.6W+0.5L(Truck) showed that the most stressed member was the diagonal with a unity check of 2.09. Table 3-13 shows a summary of the maximum unity check for each main bridge member.

TABLE 3-13. Unity Checks for LC IV: 1.2D+1.6W+0.5L (Truck)

<table>
<thead>
<tr>
<th>Member</th>
<th>Unity check</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top chord</td>
<td>0.472</td>
</tr>
<tr>
<td>Bottom chord</td>
<td>0.274</td>
</tr>
<tr>
<td>Floor beam</td>
<td>0.44</td>
</tr>
<tr>
<td>Vertical</td>
<td>0.258</td>
</tr>
<tr>
<td>Diagonal</td>
<td>2.09</td>
</tr>
</tbody>
</table>

From the analysis of the load combinations, the most critical is the first load combination.
3.5 Approximate Analysis using AISC Shapes

In order to check the reasonableness of the model and the spreadsheet, another model was created using equivalent standard steel shapes. To represent the actual top chord of the bridge, a hollow structural section was used, which can be seen in Fig. 3-19.

![Diagram of cross-sectional view of the top chord for approximate model]

**FIG. 3-19.** Cross-Sectional View of the Top Chord for Approximate Model

The actual thickness of the section used in the Approximate Model for the top chord is 0.32 in., while the thickness of the actual iron top chord is 0.325 in.

A tapered wide flange with a starting depth of 12 in. and ending depth of 24 in. was used for the floor beam. RISA’s tapered wide flange is symmetric about its longitudinal axis as shown in Fig. 3-20.
FIG. 3-20. Elevation View of Bottom Chord for Approximate Model

The Approximate Model had the same loads and conditions as Structural Model. The truss in Fig. 3-21 displays the unity checks for the first load combination, which is the dead load and the live pedestrian load. Some of the members in the figure have been omitted for clarity. From the results, it can be seen that the most stressed members are the cross bracing in the end panels and the top chord at the panel points.

FIG. 3-21. Unity Checks for LC I: 1.2D+1.6L(Ped)

The truss in Fig. 3-22 shows the interaction equation results from the second load combination, which is the dead load, wind load, and the live pedestrian load. From this load combination, the top chords are more stressed than the other members.
FIG. 3-22. Unity Checks for LC II: 1.2D+1.6W+0.5L(Ped)

The truss shown in Fig. 3-23 displays the interaction results from the third load combination, which is the dead load and the live truck load. The cross-bracing and top chord are the most stressed members.

FIG. 3-23. Unity Checks for LC III: 1.2D+1.6L(Truck)

The truss shown in Fig. 3-24 below displays the interaction results of the fourth load combination, which is the dead load, wind load, and live truck load. It can be seen that the top chord and the bottom chord are over-stressed.
3.6 Comparison of Approximate Model and Structural Model

The differences in the unity checks between the two models can be attributed to the shapes used for the Approximate Model. For instance, the equivalent steel shape used in the Approximate Model for the top chord was a hollow structural section. Even though the actual top chord is not considered a hollow structural section, it was used in the Approximate Model because RISA cannot generate hybrid shapes. The top chord in the Approximate Model used had a slightly smaller thickness than the actual top chord. This smaller thickness contributes to smaller design strengths. The smaller design strengths led to higher unity checks than what was calculated for the Structural Model.

The equivalent steel shape for the bottom chord was assumed to be a single four in. by two in. rectangular flat plate, instead of two four in. by one in. rectangular flat plates. Table 3-14 gives a comparison of the design strengths of the iron members in the Structural Model to the design strengths of the equivalent steel members in the Approximate Model.

**FIG. 3-24.** Unity Checks for LC IV: 1.2D+1.6W+0.5L(Truck)
Based on the actual configuration and shape of the verticals, an average area and moment of inertia was used in the equivalent steel model because there was not a comparable shape.

The floor beam used in the Approximate Model was doubly symmetric, whereas, the actual shape of the floor beam is a singly symmetric I-shape.

**TABLE 3-14. Strength Comparison of the Structural Model and the Approximate Model**

<table>
<thead>
<tr>
<th></th>
<th>Structural Model</th>
<th></th>
<th>Approximate Model</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Design tensile</td>
<td>Design compressive</td>
<td>Design flexural</td>
<td>Design tensile</td>
</tr>
<tr>
<td></td>
<td>strength, kip</td>
<td>strength, kip</td>
<td>strength, kip-ft</td>
<td>strength, kip</td>
</tr>
<tr>
<td>Top chord</td>
<td>305</td>
<td>244</td>
<td>98</td>
<td>275</td>
</tr>
<tr>
<td>Bottom chord</td>
<td>209</td>
<td>2</td>
<td>17.4</td>
<td>209</td>
</tr>
<tr>
<td>Floor beam</td>
<td>227</td>
<td>83</td>
<td>98</td>
<td>227</td>
</tr>
<tr>
<td>Vertical</td>
<td>59</td>
<td>n/a</td>
<td>105</td>
<td>n/a</td>
</tr>
<tr>
<td>Cross-bracing</td>
<td>11.5</td>
<td>n/a</td>
<td>n/a</td>
<td>11.5</td>
</tr>
<tr>
<td>Top lateral bracing @ 5th</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
<td>61</td>
</tr>
<tr>
<td>panel</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top lateral bracing @ 3rd</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
<td>22.9</td>
</tr>
<tr>
<td>panel</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

From Table 3-14, some of the strengths calculated are comparable to the equivalent steel shape used in RISA. Some of the strengths based on the iron shapes are higher than the equivalent steel shapes, while others are lower.

A comparison of the unity checks for the Structural Model and the Approximate Model for each type of bridge member can be seen in Table 3-15.
TABLE 3-15. Comparison of Unity Checks of the Two Models for Each Bridge Member

<table>
<thead>
<tr>
<th></th>
<th>Structural Model</th>
<th>Approximate Model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LC I</td>
<td>LC II</td>
</tr>
<tr>
<td>Top Chord</td>
<td>1.16</td>
<td>0.807</td>
</tr>
<tr>
<td>Bottom Chord</td>
<td>0.841</td>
<td>0.492</td>
</tr>
<tr>
<td>Floor Beam</td>
<td>1.29</td>
<td>0.639</td>
</tr>
<tr>
<td>Vertical</td>
<td>0.398</td>
<td>0.167</td>
</tr>
<tr>
<td>Cross-Bracing</td>
<td>1.03</td>
<td>2.1</td>
</tr>
</tbody>
</table>

It can be seen that the Approximate Model is more conservative than the Structural Model. The lower unity checks of the approximate model can be attributed to the conservative sizes that were chosen to represent the bridge in the model. The most critical load combination is the first, which is the dead load and live pedestrian load.
CHAPTER 4

LIFTING ANALYSIS

The relocation aspect of the 1882 Bridge project involved moving the bridge without disturbing the river. The plan was to lift the bridge off its abutments in one piece and set it down beside the river for later disassembly and transport. This chapter discusses several lifting methods that were analyzed for the relocation of this bridge.

The first and second methods to be discussed involve the use of a lift beam. The difference between these two methods is the location of the pick-up points on the bridge. The pick up points of the first lift beam method is located at the fourth and sixth panels, and the pick up points of the second lift beam method is located at the third and seventh panels. The third method incorporates a steel lifting frame. The fourth method requires attaching the crane cable directly to the bridge, which will be referred to as the direct lift method. These methods were evaluated based on the lifting equipment needed, required temporary bracing, and the condition of the members during the lifting process. Other methods that were considered during this analysis, including a cradling method and the use of helicopters, will also be discussed.

Because the bridge spans North River it cannot be disassembled from its current location without disrupting the creek bed, which is the reason the bridge is being moved as a whole. Ideally, the bridge should be lifted from its supports, but since the length of
this section of bridge is 144 feet and spans a creek, it is difficult to lift the bridge.

Another reason which prevents the bridge from being lifted from its supports via a crane is the location of the bridge’s center of gravity. The center of gravity is located approximately 6 feet above the bottom chord. Fig. 4-1 displays the center of gravity of the arch which is represented by a circle.

![FIG. 4-1. Location of Center of Gravity and Panel Points](image)

The location of its center of gravity is important because if the lifting cables were attached to the bridge ends, then the bridge could tilt over during the moving process. Given that the bridge cannot be lifted from its supports, the only other alternative is to lift the bridge from the top chord at a panel point using one crane instead of two. Applying the load through the top chord places part of the bottom chord in compression, and part of the top chord in tension.

The lifting methods were designed and analyzed using RISA-3D and AISC LRFD 3rd Edition. The RISA model that was used for the lifting design was the structural model mentioned in the previous chapter. The load combination that was analyzed was 1.4D plus a 25% impact load.
4.1 Lifting Methods One and Two: Lift Beam

There were two lift beam methods considered. The first was with the pick up points at the fourth and sixth panel points of the top chord, and the second method was with the pick up points at the third and seventh panel points of the top chord.

A factor of safety of five was used to design the lift beam. The lift beam was designed as an 18 feet cantilever beam supporting half the bridge weight.

4.1.1 Lift Beam Method One

In the first lift beam method, the pick up points of the bridge were the fourth and sixth panel points of the top chord. Fig. 4-2 below shows an elevation view of the lifting procedure for the first lift beam method.

![FIG. 4-2. Elevation View of Lift Beam Method One](image_url)

Cables extended vertically from the lift beam to the pick up points. The distance between the fourth and sixth panel point is 36 feet, which was the length used for the lift beam. Looking at a cross-sectional view of the bridge, the cable was at an assumed 45 degree angle from the vertical.
The distribution of forces throughout the bridge was determined using RISA-3D. The only modification made to the original structural model was the location of the supports. The supports were moved to the pick-up points of the fourth and sixth panel points to simulate the bridge being picked up from these locations. Fig. 4-3 below shows the deflected shape of the bridge utilizing Lift Beam Method One.

![Deflected Shape of Bridge During Lift Beam Method One](image)

**FIG. 4-3.** Deflected Shape of Bridge During Lift Beam Method One

The design and analysis of the lift beam was carried out using an Excel spreadsheet. The lift beam was designed as an 18 ft. cantilever wide flange with a concentrated load applied at the end equal to half the bridge weight. A shallow wide beam with sufficient lateral and torsional stiffness was chosen since the beam is assumed not to have any lateral or torsional support. Based on an unbraced length of 18 feet and required moment strength of 800 k-ft, the beam selected was a W30x99.
From Fig. 4-4, it can be seen that this lifting method puts the entire bottom chord in compression, and the top chord in tension. Negative magnitudes represent tensile forces, and positive magnitudes represent compressive forces.

![Distribution of Axial Forces in Kips for Lift Beam Method One](image)

**FIG. 4-4. Distribution of Axial Forces in Kips for Lift Beam Method One**

Table 4-1 shows the largest unity checks for the main bridge members. The calculation of the various strengths of each member was discussed in chapter three.

**TABLE 4-1. Unity Checks for Lift Beam Method One**

<table>
<thead>
<tr>
<th>Bridge members</th>
<th>Unity checks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top chord</td>
<td>0.071</td>
</tr>
<tr>
<td>Bottom chord</td>
<td>8.47</td>
</tr>
<tr>
<td>Floor beam</td>
<td>0.141</td>
</tr>
<tr>
<td>Verticals</td>
<td>0.068</td>
</tr>
<tr>
<td>Lateral bracing</td>
<td>0.05</td>
</tr>
</tbody>
</table>

RISA calculated only the vertical component of the cable reaction. The longitudinal and transverse components of the reaction were calculated in an Excel spreadsheet. The transverse component of the force was calculated to be 9.865 kips and was used to design the temporary lateral bracing for the top chord at the fourth and sixth panel points. The temporary bracing to be used was a W10X22.

WT shapes were analyzed as temporary braces for the strengthening of the bottom chord. The design size chosen was a WT5X15, which was required to withstand a compressive load of approximately 19 kips. The largest unity check of the bottom chord
being braced based on the WT-shapes is approximately 0.6. The total weight of the bridge including the bracing was calculated to be approximately 47 kips. Fig. 4-5 and Fig. 4-6 shows and elevation view and a plan view of the bridge with the bottom chords and top chords braced for the lifting method.

![Elevation View of Bracing for Lift Beam Method One](image1)

**FIG. 4-5.** Elevation View of Bracing for Lift Beam Method One

![Plan View of Bracing for Lift Beam Method One](image2)

**FIG. 4-6.** Plan View of Bracing for Lift Beam Method One

4.1.2 Lift Beam Method Two

The second method for lifting the bridge is similar to that of the first except for the location of the pick up points. The pick-up points are at the third and seventh top
chord panel points. This revised lift beam method was considered because it was assumed that if the pick up points were closer to the ends of the bridge then there would be a smaller compressive force in the bottom chord, which would reduce the amount of temporary steel bracing. It was decided to keep the lift beam at the same length as in the first method, because an increase in the length would add to the total weight which would make the lift beam an inefficient choice for lifting. However, using the same length with different pick-up points puts the cables at an angle. Initially, this was considered undesirable because it was assumed that the longitudinal component of the force from the cable would put an extreme compressive force in the top chord. When viewing an elevation view of the bridge, the angle of the cable from the vertical was assumed to be 45 degrees. Fig. 4-7 shows the lift beam with the pick up points at the third and seventh panel points.

![Fig. 4-7. Elevation View of Lift Beam Method Two](image)

The vertical component of the force from the cable was determined by RISA to be 9.865 kips. The longitudinal component of the force was calculated in an Excel spreadsheet. Based on the height of the lift beam above the supports and the angle of the
cables, the longitudinal component was calculated to be 9.18 kips. The longitudinal component was then entered into the RISA model as a compressive joint load at the pick-up points. Fig. 4-8 shows the deflected shape of the bridge with the applied joint load for this lifting method. Fig. 4-9 below shows the distribution of axial forces throughout the bridge.

FIG. 4-8. Deflected Shape of Bridge for Lift Beam Method Two

FIG. 4-9. Distribution of Axial Forces in Kips for Lift Beam Method Two
With the revised lift beam method, the bottom chord is still in compression and the outer sections of the top chord are in tension. Table 4-2 shows the maximum unity check for each type of bridge members.

**TABLE 4-2. Unity Checks for Lift Beam Method Two**

<table>
<thead>
<tr>
<th>Bridge members</th>
<th>Unity checks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top chord</td>
<td>0.303</td>
</tr>
<tr>
<td>Bottom chord</td>
<td>4.9</td>
</tr>
<tr>
<td>Floor beam</td>
<td>0.083</td>
</tr>
<tr>
<td>Verticals</td>
<td>0.044</td>
</tr>
<tr>
<td>Lateral bracing</td>
<td>0.02</td>
</tr>
</tbody>
</table>

Based on the unity checks of the bridge, it shows that the bottom chord requires temporary bracing during lifting. It was decided that WT-shapes would also be used for the bottom chord bracing for this method. The temporary bracing was required to withstand a compressive force of approximately 10 kips. The required size of the bracing to be used was a WT5x15. This was the lightest shape that could be chosen and still be considered compact. The top lateral bracing needed for the top chord at the pick up points was required to withstand an applied compressive load of approximately 5 kips. The required size of the bracing to be used was a W10x22. Fig. 4-10 and Fig. 4-11 below show the elevation view and plan view bracing needed for the top and bottom chord during lifting.
4.2 Frame Method

The frame method utilizes a rectangular steel frame to lift the bridge. This method was intended to reduce the amount of bracing needed for the bridge by picking up the bridge as close to the end panels as possible. Fig. 4-12 and Fig. 4-13 show an elevation view and a plan view of the steel frame used to lift the bridge.
The pick up points for the steel frame are at the second and eighth panel points. Even though the pick up points are closer to the end supports of the bridge, the cables are still located above the bridge’s center of gravity. The length of the steel frame was 108 feet, since that is the distance between the second and eighth panel points.

The steel frame was designed using W-shapes. The largest members of the frame, which are in the longitudinal direction of the bridge, are W14X68, and the members
parallel to the floor beams are W10X17. The steel frame was designed using an Excel spreadsheet based on the requirements of the AISC manual. The larger member of the steel frame was designed to resist the longitudinal component of the reaction from the cable. The compressive force that the cable applied was approximately 50 kips which includes the safety factor of five. Based on an unbraced length of 54 feet, the compressive strength of the member was calculated to be approximately 62 kips. The W10X17 was designed to resist the transverse component of the cable and to also brace the large frame member. The force that the cable applied to the W10X17 was approximately 9.5 kips, which includes the required safety factor. The compressive strength of this member was calculated to be approximately 13.5 kips.

Fig. 4-14 displays the deflected shape of the bridge for the frame method.

![FIG. 4-14. Deflected Shape of Bridge for Lifting Frame Method](image)

Fig. 4-15 below shows the distribution of axial forces throughout the bridge during lifting. The analysis of the lifting method showed that the bottom chord would not buckle except at the end panels, which can be seen in Table 4-3.
FIG. 4-15. Distribution of Axial Forces in Kips for Frame Method

TABLE 4-3. Unity Checks for Lifting Frame Method

<table>
<thead>
<tr>
<th>Bridge members</th>
<th>Unity checks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top chord</td>
<td>0.122</td>
</tr>
<tr>
<td>Bottom chord</td>
<td>2.055</td>
</tr>
<tr>
<td>Floor beam</td>
<td>0.083</td>
</tr>
<tr>
<td>Verticals</td>
<td>0.012</td>
</tr>
<tr>
<td>Lateral bracing</td>
<td>0.05</td>
</tr>
</tbody>
</table>

The bracing needed for the bottom chord was at the end panels of the bridge. The required bracing needed to withstand the compressive force of 4.4 kips was a WT5X15. Temporary bracing was not needed for the top chord during this lifting method. Since the cables extending from the top chord were vertical, any unexpected compressive force that was placed on the top chord was assumed to be resisted by the existing bracing. Fig. 4-16 and Fig. 4-17 show the bracing required for this lifting method.
4.3 Direct Lift Method

The direct lift method utilizes four cables, with each cable extending from the pick-up points at an assumed 45 degree angle to the crane. Unlike the other methods, this method does not require the use of any frame or beam to lift the bridge. The direct lift
method has its pick-up points at the second and eighth panel points. Fig. 4-18 shows the cables used to lift the bridge.

![Diagram of Crane Hook and Bridge]

**FIG. 4-18.** Elevation View of Direct Lift Method

This method was initially disregarded because it was assumed that the angle of the cables would put an extreme compressive force in the top chord. The longitudinal and transverse components of the resultant force in the cable were calculated using a spreadsheet. The longitudinal force of the cable was entered into RISA as a joint load of 9.78 kips. The transverse component of the cable was 1.81 kips, which was the required compressive force that the top lateral bracing was required to resist during this lifting procedure. Fig. 4-19 shows the deflected shape of the bridge with the applied joint loads. Fig. 4-20 shows the distribution of the axial forces throughout the bridge. Table 4-4 shows the unity checks for the main bridge members.
FIG. 4-19. Deflected Shape of Bridge During Direct Lift Method

FIG. 4-20. Distribution of Axial Forces in Kips for Direct Lift Method

TABLE 4-4. Unity Checks for Direct Lift Method

<table>
<thead>
<tr>
<th>Bridge members</th>
<th>Unity checks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top chord</td>
<td>0.123</td>
</tr>
<tr>
<td>Bottom chord</td>
<td>2.104</td>
</tr>
<tr>
<td>Floor beam</td>
<td>0.082</td>
</tr>
<tr>
<td>Vertical</td>
<td>0.012</td>
</tr>
<tr>
<td>Top lateral bracing</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Temporary bracing was needed for the end panels of the bottom chord for this lifting method. It was decided to brace the bottom chord with wide flange shapes instead of tee shapes because those shapes can provide a better connection. The bracing chosen
for the bottom chord was a W10X22 which had to withstand a compressive load of approximately 2.5 kips. Fig. 4-21 and Fig. 4-22 show an elevation view and a plan view of the temporary bracing needed for this lifting method.

**FIG. 4-21.** Elevation View of Bracing for Direct Lift Method

**FIG. 4-22.** Plan View of Bracing for Direct Lift Method

4.4 Discussion of Results

All lifting procedures require temporary steel bracing for the bottom chord, and most lifting procedures require bracing for the top chord between the pick up points. Table 4-5 displays a comparison of the equipment needed for each lifting method. Table 4-6 compares the maximum unity check of the members for each lifting method.
TABLE 4-5. Comparison of Required Equipment for Each Lifting Method

<table>
<thead>
<tr>
<th>Lift method</th>
<th>Lifting equipment required</th>
<th>Transverse bracing at pick points</th>
<th>Bottom chord bracing</th>
<th>Weight of bracing and equipment, k</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lift beam I</td>
<td>W30x99</td>
<td>W10X22</td>
<td>WT 5X15</td>
<td>11</td>
</tr>
<tr>
<td>Lift beam II</td>
<td>W30X99</td>
<td>W10X22</td>
<td>WT 5X15</td>
<td>11</td>
</tr>
<tr>
<td>Frame</td>
<td>4-W14X68</td>
<td>n/a</td>
<td>WT 5X15</td>
<td>16.7</td>
</tr>
<tr>
<td>Direct lift</td>
<td>Cables</td>
<td>W10X22</td>
<td>W10X22</td>
<td>1.5</td>
</tr>
</tbody>
</table>

TABLE 4-6. Comparison of Unity Checks for Each Lifting Method

<table>
<thead>
<tr>
<th></th>
<th>Top chord</th>
<th>Unbraced bottom chord</th>
<th>Floor beam</th>
<th>Vertical</th>
<th>Top lateral bracing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lift beam I</td>
<td>0.071</td>
<td>8.47</td>
<td>0.141</td>
<td>0.068</td>
<td></td>
</tr>
<tr>
<td>Lift beam II</td>
<td>0.303</td>
<td>4.9</td>
<td>0.083</td>
<td>0.044</td>
<td>0.02</td>
</tr>
<tr>
<td>Frame</td>
<td>0.122</td>
<td>2.055</td>
<td>0.083</td>
<td>0.012</td>
<td>0.05</td>
</tr>
<tr>
<td>Direct lift</td>
<td>0.123</td>
<td>2.104</td>
<td>0.082</td>
<td>0.012</td>
<td>0.05</td>
</tr>
</tbody>
</table>

The first lift beam method shows an extreme compressive force being applied to the bottom chord, which is a result of the location of the pick up points. This large compressive force occurs because the top chord, after the location of the pick up points, is not being supported, it is hanging freely. This part of the top chord that is hanging is in tension, and the bottom chord is in compression. Given the weight of the lift beam and temporary steel bracing required for the lift, this method is an undesirable choice.

To reduce the amount of compressive force in the bottom chord, the location of the pick up points had to be moved closer to the ends of the bridge. The second lift beam method utilized different support locations than the first method. For this method, the cable was to be attached to the third and seventh panel point, in order to produce a smaller compressive force in the bottom chord. The analysis of this method showed that the compression in the bottom chord was reduced by approximately 42% by moving the
pick up points. However, the analysis also concluded that the amount of temporary bracing would not be reduced, because every member of the bottom chord was overstressed.

The frame method was developed to move the location of the pick up points even closer to the end of the bridge and also to keep the angle of the cable from the bridge to the frame at a vertical. The results of this procedure conclude that the bottom chord is in compression only at the end panels of the bridge. For this method, the amount of temporary bracing has been reduced. Even though the bridge withstand this method more than the other two lift beam methods, the size and length of the lifting frame makes this method an inefficient choice.

As previously mentioned, the direct lift method was initially disregarded. Once the forces on the bridge due to this method were analyzed in RISA, it shows that there is not a significantly large compressive force being applied to the top chord. Compared to the frame method, the forces in the top chord are higher as expected. The bottom chord is overstressed only at the end panels as with the frame method. The advantage this method has over the frame method is that the only equipment needed besides the temporary bracing are the cables.

A comparison of the above two tables show that the ideal lifting method is the Direct Lift Method. This method has the advantage of being lighter than the other methods, and it also does not have the extensive rigging equipment as the other methods did.
CHAPTER 5
FINAL LOCATION DESIGN

This chapter discusses the design of the wood decking, abutments, and approaches for the bridge at its final location. The purpose in designing these components was to obtain cost estimates for the bridge relocation project.

The design of the decking system along with the estimate will be presented first, followed by the design of the abutments and its cost estimate, and, finally, the approaches with cost estimates will be discussed.

5.1 Deck Design

Designing the decking system for the bridge included the design of the stringers and decking. The wood to be used for the decking system was assumed to be Southern Pine No. 2 with material properties from the 2005 National Design Specification (NDS) from the American Forest and Paper Association.

The stringers run parallel to the bottom chord and the decking is perpendicular to the bottom chord. One of the assumptions in designing the stringers was that the decking distributes some of the load to the other stringers. The stringer design was based on the truck load and checked against the pedestrian load. The stringers and decking were designed for bending and shear, with the controlling load combination being the truck
load. The front and rear axle of the maintenance vehicle was positioned over the stringer to produce a maximum moment. The maintenance vehicle was assumed to be a 10,000 pound truck. The rear axle carried 60% of the total vehicular weight and the front axle carried 40% of the total vehicular weight.

A dynamic analysis of the truck load on the decking was not necessary because the slow speed of the truck justifies neglecting it. The design required a stringer size and spacing of 4x12's at 12 inches on center. The decking to be used was 2x6, and the size of the running boards was also 2x6. The wood decking system can be seen in Fig. 5-1.

![Decking System](image)

**FIG. 5-1.** Decking System

The wheel loads were distributed laterally to adjacent stringers based on the relative stiffness of the stringers and decking. The design was checked using Timber Bridges: Design, Construction, Inspection, and Maintenance by the United States Department of Agriculture (USDA). This method required a minimum of 4” thick decking and showed that 4x12 stringers spaced at 22 inches on center were adequate. A
check was made on the initial design using USDA’s method, and it was determined that the size of the decking should be 4x10 and the size and spacing of the stringers were 4x12’s at 22 inches on center.

The estimates that were obtained for the wood were based on the first design method and include only the materials. An estimate received from Harper Chambers Lumber Company, Inc. was $23,093.52, and another estimate received from 84 Lumber Company was $19,599.20. These estimates do not include blocking for lateral bracing or any connection materials. An approximate average estimate of $20,000.00 was used for the cost of the decking system.

5.2 Bridge Abutment

The length of the abutment was set equal to the width of the bridge which was 21 feet. The abutment was designed according to the American Concrete Institute 2005 code. The abutment was designed to support the bridge while also acting as a retaining wall. The abutment wall was designed as a column where it supports the top chord and end plate, and where it supports the soil, it was designed as a retaining wall. The concrete abutment was designed based on a normal weight concrete with a compressive strength of 4000 psi. The footing was designed as a strip wall footing. Based on the structural fill material of the levee, the gross allowable soil pressure assumed was 4000 psf. Fig. 5-2 and Fig. 5-3 show the cross-sectional view and plan view of the abutment.
An estimate for the concrete abutments was received from SOBCON, a local subcontractor specializing in concrete projects. Each abutment was priced at $11,000.00. The cost of this estimate includes, concrete, footing excavation, forms, and rebar, but it does not include site work and grading, fill material, anchor bolts, stake off, and testing.
5.3 Approaches

Based on the final design of the decking system and the abutment, the height of the bridge above the natural soil grade was approximately 57 inches. Since the bridge will be for public use, it must be ADA compliant. In order for it to be ADA compliant, the bridge must be easily accessible for wheelchairs. A ramp will be formed from fill material at a 1:12 slope, which will lead to the top of the decking. A 1:12 slope yields a ramp length of approximately 57 feet. The width of the walkway is 6 feet and the width of the bridge is 18 feet. The ramp will be built up to equal the width of the bridge. The fill material will be supported laterally by Keystone wall units. Keystone units are more expensive than concrete, but were considered more aesthetically pleasing by project team members from Friends of Historic Northport. Fig. 5-4 shows an elevation view of the Keystone wall.

![Fig. 5-4. Elevation View of the Keystone Approach](image)

Keystone walls taller than 4.3 feet require reinforcing fabric. The reinforcing fabric was designed according to the manufacturer’s specifications. The friction angle of
the soil was assumed to be 30 degrees. The type of soil was assumed to be silty sand, and the surcharge was taken as 250 psf. Based on this assumed information, the amount of fabric needed at the tallest portion of the wall, which is approximately 9.5 feet, are three layers of fabric extending out from the wall into the soil approximately 6.5 feet. The amount of fabric needed for the wall at the shorter end, which is approximately 3.5 feet, was one layer of fabric extending into the soil at 5 feet.

An estimate was received from Willcutt Block for the Keystone wall. There were approximately 3100 units needed for the approach which were priced at $4.05/unit. The reinforcing fabric for the approach was priced at $65 for each 56 square feet of material needed. The total amount of reinforcing fabric needed was approximately 4800 square feet. The total cost of the wall with fabric was approximately $18,000.00.

The total cost of the materials for the wood decking, abutments, and approaches was approximately $60,000.00. The 1882 Bridge Relocation Project is expected to incur other costs, but they are considered to be outside the scope of this thesis.
CHAPTER 6
SUMMARY AND CONCLUSIONS

This chapter summarizes the work conducted for this thesis, including the properties of the wrought iron, analysis of the bridge for expected loads at its new location, and the design of the lifting methods. Proposed future research includes an analysis of the bridge connections, accurate measurement of the modulus of elasticity of wrought iron, taking accurate measurement of the dimensions of the top lateral bracing of the bridge, and also using non-destructive testing procedures to establish if the bridge members have any flaws or fatigue cracks.

6.1 Conclusions

Samples from the 1882 Bridge included four cylindrical samples of the cross-bracing, and two rectangular samples of the floor beam. The tensile tests conducted on the sample conclude that the tensile strength ranged from 41,000 psi to 54,000 psi. The yield stress of the samples ranged from 29,000 psi to 38,500 psi. The ductility of the samples was measured by the reduction in area. The reduction in area of the samples ranged from 16%-22%. Based on earlier specifications for wrought iron, the acceptable standards called for a tensile strength of 55,114 – 60,045 psi and a 25% reduction in area in a tensile test. Therefore, the tested samples indicate the presence of brittle iron and that the tensile strength is lower than the published acceptable standards. Gordon and
Knopf point out that material testing is not always the best way to determine material properties, because the samples that are taken are not always representative of the structure.

A metallurgical analysis was performed on the samples to ascertain its chemical composition. The base material was 98% iron and 2% oxygen. A typical slag inclusion was 53% iron, 27% oxygen, 8% phosphorous, and 6 other elements. There were many elements, including iron, found in the slag inclusion, while the other table shows only the presence of the base material, iron, and oxygen.

Research by Gordon and Knopf establishes that phosphorous can reduce the ductility of wrought iron, and can also give inadequate toughness in load-bearing structures if the average content exceeds 0.3%. The phosphorous content found in the sample could contribute to the samples not meeting the minimum specification for ductility. Manganese was also found in the slag inclusion, which is common in wrought iron, but it does not influence the strength and ductility as much as phosphorous. Since the manganese content in the samples was less than 1%, its presence in the slag does not affect the properties of the sample. Silicon was also detected in the samples, but it is not considered a source of embrittlement according to Gordon and Knopf.

The bridge was analyzed for loads expected at its new location on the walking path atop the levee in Northport. The forces in the bridge members due to the loads were calculated using RISA, a structural analysis program. The member strengths and unity checks were calculated using AISC LRFD equations and an Excel spreadsheet.

The unity checks were within design requirements for all load combinations except LC I: 1.2D+1.6L(Ped). A large pedestrian load of 85 psf with no live load
reduction was used as specified by AASHTO. The unity checks and live load reduction factors for the three overstressed members are shown in Table 6-1. When a live load reduction factor is included, only the floor beams have a unity check greater than one.

**TABLE 6-1. Unity Checks for LC I and Live Load Reduction Factors**

<table>
<thead>
<tr>
<th></th>
<th>Unity check</th>
<th>Tributary area, A_t</th>
<th>Reduction factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top chord</td>
<td>1.16</td>
<td>1420</td>
<td>0.53</td>
</tr>
<tr>
<td>Floor beam</td>
<td>1.29</td>
<td>360</td>
<td>0.8</td>
</tr>
<tr>
<td>Diagonal</td>
<td>1.03</td>
<td>180</td>
<td>1</td>
</tr>
</tbody>
</table>

The structural analysis for the bridge at its future site showed that the bridge should perform well. The most critical member is the floor beam under the pedestrian load. Since the 1882 Bridge is not considered a critical structure, the fact that the floor beam’s check exceeds one is tolerable. If the floor beams should fail under the loading given in the first combination, no loss of life would be sustained since the bridge will be sitting on top of the levee’s walking path.

Four lifting schemes were analyzed to determine which one would be the most efficient method, while also making sure that the bridge is not too stressed. The two methods using lift beams required an excessive amount of bracing for the bottom chord. The third method, which involved constructing a lift frame, required less bracing on the bridge. The fourth method, which involves directly attaching the cable to the top chord, is the preferred method because the bridge requires minimal bracing.

The design of the final location was used to estimate the cost of the project, which included the wood decking system, the bridge abutments, and approaches. The lateral distribution of wheel loads was based on relative flexibility of the stringers and decking. Using this method, the required size and spacing for the stringers were 4x12’s at 12
inches on center, and the deck size was 2x6’s. This design was checked with the procedure used in Timber Bridges by USDA. This method required a 4” thick decking, and the size and spacing of stringers were 4x12’s at 22 inches on center. Based on the design of the first method, the cost of the estimate was approximately $20,000.00. This cost includes materials only.

The bridge abutments were designed according to the ACI 2005 code. The wall was designed to act as a column where the supports of the bridge will rest, and it was designed to act as a retaining wall to resist the lateral earth load. Each abutment was estimated to be approximately $11,000.00.

The approaches for the bridge were designed for a keystone block wall. A keystone wall was used because it is more aesthetically pleasing than concrete. The estimate for the keystone wall and the reinforcing fabric was approximately $18,000.00.

The total cost of the decking system, abutments, and approaches are approximately $60,000.00, which does not include any labor cost.

6.2 Future Research

There are several areas of this investigation that could be explored in more depth, but was considered, at the time, to be outside the scope of this thesis. They include investigating the strength of the connections, measuring the dimensions of the top lateral bracing of the bridge, conducting more accurate tensile tests, and performing a non-destructive test on the bridge.
The connections were not analyzed because many areas of the bridge were not easily accessible. The connections should be inspected when the bridge is disassembled for transportation and the capacities analyzed.

Assumptions were made regarding the sizes of the top lateral bracing, because, like the connections, they were difficult to access. When the bridge is disassembled for transportation they should be measured to determine accurate strength capacities.

A tensile test should be performed on another sample of the bridge using an extensometer to accurately determine its modulus of elasticity. An extensometer was not used on the tensile tests of the samples, so an accepted value of 28,500 ksi was used as the modulus of elasticity for wrought iron.

A non-destructive test, such as an ultra-sonic test could be conducted on the bridge. This type of test will determine if there are any fatigue cracks in the bridge, and also if there are any inconsistencies in the wrought iron of the members. One of the assumptions of this thesis is that the wrought iron is in good condition throughout the bridge.
REFERENCES


Personal Communication with NNW, Inc on July 24, 2006 (e-mail).
