Preservation of Early Wrought Iron Trusses: the 1848 Roof of the Cochituate Gatehouse

by

Lori Ferriss

Submitted to the Department of Architecture
In Partial Fulfillment of the Requirements for the Degree of

Bachelor of Science in Art and Design

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ABSTRACT

This thesis investigates the historic significance, structural condition, and preservation challenges of the wrought iron roof trusses of the Cochituate aqueduct’s inlet gatehouse as the possible earliest surviving example of their type in the United States. Through an examination of the existing structure and archival documentation of the structure’s history from archived documents, this project establishes the necessity for the structure’s preservation and offers solutions for its future restoration.

Thesis Supervisor: John Ochsendorf
Title: Associate Professor of Building Technology
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A. Buckling Calculations
Chapter 1: Introduction

1.1 Objectives

This thesis is an attempt to understand the historic significance, structural design, and current condition of the original 1848 wrought iron trusses supporting the roof of the Cochituate aqueduct’s upper gatehouse, which are possibly the oldest surviving entirely wrought iron roof trusses in the United States. An understanding of the roof’s importance to the history of building technology and American engineering will highlight the importance of preservation of the original trusses. A structural analysis will provide insight into the performance and possible changes in usage of the gatehouse roof over the course of its lifetime, as well as into the contemporary design and construction methods of the period. Additionally, the structural analysis and site inspections will provide a framework for future preservation work on the gatehouse and roof. Design considerations and suggestions will take into account the trusses’ unique place as a historic example as well as the building’s evolution over 150 years and its current state as part of the Cochituate State Park.
1.2 Literature review and archival research

While there is a large amount of literature detailing the technological history of wrought iron and the use of iron in architecture, there are very few texts regarding American production and development of wrought iron. As many principle advances in the field took place in Europe, the majority of texts used as references on wrought iron building history for this thesis focus on European buildings and figures.

For information on construction and repairs to the gatehouses, the existing primary sources known at this time are reports and financial records from the Boston Metropolitan Water Board, as well as journal articles and reference texts from the time. Additionally, the *History of the Introduction of Pure Water into the City of Boston* by Nathaniel Bradlee in 1868 contained some details such as the names of several of the companies involved in the aqueducts construction. These texts have provided the majority of information regarding the timeline of the aqueduct and history of its use.

Historic images also provided insight into the state of the gatehouse over the period of its use. In particular, a book of drawings of the aqueduct

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dating from the time of its construction was located in the Massachusetts Water Resources Authority (MWRA) library, providing multiple renderings of the gatehouse in section and elevation that were used as a comparison to its current condition. The MWRA archives also held a drawing from 1903 showing the original truss structure, which has provided measurements and a detailed understanding of the trusses’ forms.²

1.3 Site Inspection

All information about the condition of the trusses and the gatehouse since 1915 was gathered through inspection of the site. A hole in the ceiling of the gatehouse provides a limited amount of access to the trusses, allowing for the collection of measurements and images of the roof trusses. The assessment of the condition of the trusses and the building is also based on information gathered from site visits.

1.4 Structural Analysis

Several methods of analysis were used to complete a structural evaluation of the roof’s truss system. Basic hand calculations by graphical analysis were used to determine an initial approximation of each truss’

² Drawing by W. W. Patch, 1903. MWRA Archives.
behavior and load-bearing capacity. Arcade, a non-linear structural analysis
program developed at the University of Virginia, was then used to confirm
the previous results and to model alternative loading and construction
scenarios.\(^3\) The combination of results from these methods was then verified
based on correspondence to physical observations and historical facts.

\(^3\) http://www.arch.virginia.edu/arcade/ 2008.
Chapter 2: Cochituate Aqueduct and Gatehouses

2.1 Construction and history

The Cochituate aqueduct was constructed from 1846 to 1848 as the first public project to bring water into the city of Boston (Figure 2.1). The project, costing approximately three million dollars at the time, consisted of the construction of an aqueduct that carried water from Long Pond (named Lake Cochituate during the project) to the Brookline reservoir and into Boston, as well as “various works of masonry...among [which] are the gatehouses at Cochituate Lake and at the Brookline reservoir.”

The construction of this system was significant not just as a landmark for the City of Boston, but also as one of the earliest efforts of the type in the United States. Due to the limited precedent for this type of project, its undertaking involved parties with a range of specialties and industrial contributions from the surrounding region. Iron for the project, for example, was supplied by manufacturers as near as the South Boston Iron Works, and as far as the West Point Foundry and Colwell & Co. in Philadelphia.

Ellis Sylvester Chesbrough (1813-1886) acted as chief engineer on the Western division of the aqueduct, which included both gatehouses, with

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5 Ibid.
John Bloomfield Jervis (1795-1885) serving he role of consulting engineer during the early stages planning. Jervis had extensive experience as a railroad and canal engineer. In 1817 he started working on the Erie Canal, and would go on to act as the chief engineer of the Delaware and Hudson canal project. He also designed the Croton aqueduct in New York in the 1830s. Chesbrough began his engineering career working as a surveyor for railroad companies in 1828. After completing work on the Cochituate aqueduct’s construction, he would become Boston’s first City Engineer, and would go on to develop Chicago’s water and sewage system.

Within a decade of the project’s completion, Boston’s water needs began to outgrow the capacity of the aqueduct system. In 1859, the first expansion of the project was completed. In 1870, the Chestnut Hill reservoir was constructed to supplement the city’s water supply, intercepting flow from the Cochituate aqueduct, and later bypassing the Brookline reservoir altogether with a new water main. In 1903 the Brookline reservoir, including the lower gatehouse, was sold to the city of Brookline for park use. Cochituate State Park was founded in 1947, encompassing the inlet

9 Metropolitan Water Works Annual Report, 1870.
gatehouse and Lake Cochituate. In 1951, the aqueduct and reservoir were finally abandoned as a functioning system.

2.1 View of the Water Celebration, On Boston Common, October 25th 1848. Lithograph by P. Hyman and David Bigelow. www.archives.gov/research/american-cities/
The opening ceremony drew a crowd of 100,000 people to the Commons.

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2.2 The Brookline Gatehouse

The lower gatehouse of the Cochituate aqueduct is situated on the Brookline reservoir (Figure 2.2). It was designed by Charles E. Parker to serve both functional as well as recreational purposes. In addition to controlling the flow of water into the Brookline reservoir, the gatehouse was also the entrance to the reservoir grounds, and can be seen in early renderings of the aqueduct project.

Figure 2.2 View of the Brookline Gatehouse, Boston Public Library
The building is constructed of local granite, and houses a wrought iron truss system that originally held a rare self-supporting iron sheet roof. It also contains a unique set of cast iron stairs that may also be of historic significance. In the absence of information regarding the structure’s engineer, it can be assumed that Chesbrough was responsible for the design of the roof as the overseeing engineer of the building’s construction.

The roof structure of the Brookline gatehouse consists of twenty-one uniform wrought iron trusses (Figure 2.3). They are more closely spaced than those of the Cochituate gatehouse, and were produced in a more industrial, standardized manner. Each truss is riveted together, having likely been prefabricated and then assembled on-site. The trusses rest on the top surface of the building’s granite walls, and are held to each other with tie beams down the center of the roof.

Unlike other components of the aqueduct, the Brookline gatehouse remained occupied long after its period of use as a functioning gatehouse. In 1926, the building was renovated by the Brookline Sportsmen’s Club, who used it through the 1960s.
2.3 The Cochituate Gatehouse

The upper gatehouse of the aqueduct (called the Cochituate gatehouse), which is located in Wayland in what is now the Cochituate State Park, is approximately 30’ x 40’ and was constructed with the aqueduct between 1846 and 1848. The building was designed in a Mannerist-Renaissance style, seemingly monumental and elegant considering its utilitarian program, however the architect remains unknown. Like the Brookline gatehouse, its walls are of local granite and brick, and its roof is supported by original wrought iron trusses. During its years of use, it served to control the flow of water from Lake Cochituate into the aqueduct (Figure 2.4).
The Cochituate gatehouse was significantly renovated or repaired at least three times throughout its period of use. In 1859, Lake Cochituate was deepened to expand the system’s capacity, resulting in the gatehouse being raised 4’ 8”. As part of this $19,000 undertaking, the gatehouse was taken down and reconstructed on its original plan and foundation. While the Water Board’s report specifies that the floor and walls were raised to stand exactly as they had before, it does not mention whether the original roof was retained, or what other alterations, such as additional stairs, were made to the structure to account for its increased elevation. Since there is no documentation from this time indicating the purchase or design of a new roof as exists from other periods of renovation to the building, it is likely that the gatehouse kept its original roof.

The next known renovation to the structure occurred in 1903 as part of nearly $104,000 of improvements that included the excavation and cleaning of areas in Natick and Wayland adjacent to the gatehouse. The earliest available set of drawings detailing the roof’s original truss structure resulted from measurements taken during this period of construction (Figure 2.5). This special attention to the roof may indicate that the current tie beams,

strong points, or other retrofits were added to the truss at this time. The drawing does, however indicate that the sistered wooden beams were already in place by 1903 in a note reading “the compressive stress is evidently carried mainly by 3”x4” roof timbers which are beside the rafters.”

In 1915, the gatehouse’s roof was replaced. The Metropolitan Water Board’s report from that year says that the previous tin roof had been blown off, having been in service for many years. This is the only known documentation of the gatehouse’s roof material, and indicates that a tin roof had been in place since at least the late 1800s.

2. 4 1849 drawing of the Cochituate gatehouse in elevation

14 Drawing by W.W. Patch. 1903
15 “Metropolitan Water Board Annual Report.” 1915
2.4 Description and characteristics of the roof trusses

The roof of the Cochituate gatehouse is unique within the whole of the aqueduct project in its use of wrought iron. It consists of five wrought iron Howe trusses and a series of wrought iron rafters attached to a wall plate atop the building’s walls. Because the building is covered with a hip roof, two types of trusses are used; there are three triangular trusses in the center, with a hip truss on either end. While the roof truss at the Brookline gatehouse is constructed with early forms of standardized, industrial
techniques, the Cochituate truss is constructed in a much more craftsman-like manner. Elements within each truss vary in cross-section from simple rectangular straps, to circles that transition to become orthogonal at the joints, to parallel chords (Figure 2.7). Additionally, the trusses are assembled in varying manners, including versions of mortise and tendon connections and hooks (Figures 2.8, 2.9). Additionally, the truss members on average have a much greater cross-sectional area than at the Brookline gatehouse as is consistent with there being fewer trusses.

Through the current bead board ceiling, a series of four iron rings are visible along the centerline of the roof. These strong points are each aligned with one of the original trusses. Additionally, there is an I-beam crane rod above one of the subsequently added tie beams (Figure 2.10). This crane rod aligns with a slit in the wall below it where a gate or screen might have sat.
Figure 2.7 Parallel chord and circular elements

Figure 2.8 Attachment of purlin to triangular truss

Figure 2.9 Mortise and tenon joint at meeting of wall plate and truss

Figure 2.10 Crane rod
Chapter 3: Wrought Iron as a Material in Historic Structures

3.1 Development of wrought iron as a structural material

The first references to the use of iron in construction date back to antiquity with references made by Homer and Hesiod. While wrought iron has been used as a structural material in the west since about 450 B.C., and longer in other countries such as India and China, it did not become widely produced until the 1760s to 1830s due to financial and technical constraints. Beginning from this period, iron became an integral building material, and was not replaced by structural steel until the late 1800s, several decades after the invention of the Bessemer converter in 1855.

In 1783-4, Henry Cort introduced the puddling and rolling processes in England, allowing for the production of more uniform iron in more standardized forms. These new technologies, combined with the invention of the steam engine, increased the effectiveness of iron production in all stages. A series of further developments, including the invention of the hot-blast process by Neilson in 1824, made wrought iron a more affordable and convenient structural building material. Throughout the early decades of the century, standardized forms such as tees and angles were developed to

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16 Faribairn, William, 3
17 Sutherland, R.J.M. “Pioneer…” 99
18 Fairbairn, William
aid the design of iron structures. In the 1840s, wrought iron members in forms resembling those used today began to emerge; in 1844, Kennedy and Vernon developed the “bulb tee,” which was followed by the manufacture of the I-beam in France circa 1848.¹⁹

The 1840s were a period of experimentation and development not only in the production of wrought iron, but also regarding its structural role in design and construction. During this decade, for example, William Fairbairn (1789-1874) and Eaton Hodgkinson (1789-1861) conducted systematic tests on tubular girders – composite structures comprised of plates, angles, tees joined by rivets. Shortly thereafter, Richard Turner (1798-1881) experimented with the idea of a wrought iron purlin that could be used to tie ribs together horizontally.²⁰

Possibly the earliest standing wrought iron truss roof was the roof of Euston Station in London, which was constructed between 1835 and 1837. Wrought iron use continued to develop throughout the century, with the ultimate achievement of construction in wrought iron marked by the design and erection of the Eiffel Tower in 1889. After that point, steel quickly replaced iron as a structural building material for its ease of production and similarly high tensile capabilities.

¹⁹ Diestelkamp, Edward, 90
²⁰ Ibid 92
3.2 History of wrought iron in the United States

Iron work was first brought America by European settlers. During the seventeenth century, iron was used primarily as hardware. By the late eighteenth century, it and was formed into balconies, railings, and other types of applied ornamentation.$^{21}$ With the emergence of early industrial technologies around the turn of the nineteenth century, iron was increasingly used in construction as an answer to the need for fireproofing in industrial buildings as well as the need for more light in interior spaces, among many other reasons.

In around 1830, the Bond Building on Merchant’s Row in Boston was one of the first buildings with structural iron elements. At the same time, the South Boston Iron Works was conceptualizing new forms of iron-framed housing projects. They were also the company that fabricated the wrought iron frame of Black Rock Lighthouse on the Long Island Sound in 1843, one of the earliest of its kind.$^{22}$

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$^{21}$ Geerlings, Gerald K. 145
Other than the Cochituate aqueduct's gatehouses, the earliest known surviving wrought iron roof trusses are located in the St. Louis Courthouse from 1851.

3.3 Properties and methods of evaluation for historic iron

Wrought iron created before the last decades of the nineteenth century has been found to vary greatly in ductility while remaining fairly consistent in strength. The two most common causes of low-ductility are large percentages of phosphorous or carbon and a poor distribution of slag fibers. Because of its composite nature, namely its inclusion of the abovementioned metal deposits and residual slag fibers from production, wrought iron cannot be evaluated for structural soundness like other materials such as steel. Both metal impurities and slag fiber distribution, however, can be detected through means other than mechanical testing.\(^23\)

The earliest documented method of evaluating wrought iron was developed by Thomas Telford (1757-1834) in 1814 to fill his need for reliable data as he designed a suspension bridge in Runcorn, England. In the United States, the first testing of wrought iron properties was requested by Congress to address exploding boilers. Testing was carried out at the

Franklin Institute in 1832-33 by Walter Johnson and Benjamin Reeves (1779-1844). These tests were the first to show that inadequate ductility could undermine the structural integrity of the material. During the course of the testing, Johnson and Reeves assessed the properties of samples from three major American wrought iron producers: Ellicott, Lukens, and Schoenberger. They found that while all three manufacturers produced iron within the same strength range, the latter two lacked the necessary ductility in some samples.

It was not until 1856 that Robert Mallet published the theory that structural soundness of iron depended on toughness, the product of ductility and strength. Several decades later, serious testing of ductility compared to strength became more prevalent. In the 1870s, when the production process for iron had existed long enough in the United States for the techniques to be mastered, A. L. Holley collected data for ship building; he found that the highest grades of iron available at the time had ductility values ranging anywhere from 26% to 54% area reduction on samples with an average strength of 365MPa. The first numerical requirements for ductility were specified for the construction of the International Bridge in Buffalo, NY in 1873, which stipulated a tensile strength of 380-414 MPa and a ductility of at least 25% reduction in area.
The presence of carbon and phosphorous as well as inclusions of nonmetallic fibers can detract from the performance of wrought iron. Nonmetallic fibers left in the material from the production process can cause a decrease in compressive strength compared to tensile strength along the axis of the fibers. Additionally, these fibers are mainly iron-silicon-oxygen composites (fayalite or similar materials) that are more brittle than iron, and therefore reduce the overall ductility of the material. Carbon and phosphorous also have an effect on the strength and ductility of wrought-iron, although carbon is not seen as much in iron created by puddling, the predominant technique of the early 1800s.24

These factors in the lasting performance of historic wrought iron necessitate the use of different evaluation techniques. Wrought iron as a material does not deteriorate over time; age will not in itself affect its structural integrity as it would in steel. A simple inspection of the structure can detect cracks or other defects that may cause general instabilities. Traditional ductility testing is expensive and destructive because of the standard sample-size required of load-bearing members, however,

metallographic testing can reveal quantities and distribution of slag, carbon, and phosphorous on site or with small, nondestructive samples.\textsuperscript{25}

\textsuperscript{25} Ibid.
Chapter 4: Structural Analysis

4.1 Assumptions

Given the complex nature of the roof’s design and construction, simplifications were made throughout the structural modeling and calculations in this project. The first assumption made is that the loads on the roof are distributed equally over the five trusses, A-E as marked in Patch’s drawing (Figure 4.1). While the trusses are spaced differently based on their location in the roof, this is roughly compensated for by the variability in roof area per length of the wall that results from a hip roof; the trusses at the edges of the roof are spaced further apart, but the roof is also a smaller surface at this point.

Additionally, for initial calculations, it was assumed that all loads were applied symmetrically at three joints in the trusses as indicated (Figure 4.1), and that truss elements acted only in axial compression or tension. Dead loads were taken to be the weight of the wrought iron trusses and rafters (6050 lbs) plus the weight of a tin roof (2550 lbs), totaling 8600 lbs. Snow was the primary live load taken into consideration, assumed to be thirty pounds per square foot (total of 38,000 lbs applied uniformly over the roof). The total load (P) is divided into five equal values as shown below.
In determining the maximum stresses permissible in each truss member, a yield stress of 27,000 psi and Young’s modulus of 30,000,000 psi were assumed for wrought iron. The buckling capacity of each member was also a factor in determining the maximum loads for the roof. Given the complex methods and unknown condition of the joints, critical buckling loads were estimated using Euler’s formula for a single supported and double clamped beam to provide a range at which the members might fail in buckling (Appendix A).

4.2 Graphical analysis

A graphical analysis of each truss was completed as an initial estimate of internal forces for each type of truss in the roof. Using this method, scale drawings of each truss were used to create a scaled force polygon from which the force in each member could be measured (Figure 4.2). This was a
common structural design and analysis tool in the late 19th century for iron trusses, but was likely not used in the 1840s in Boston.\textsuperscript{26}

For trusses B-D, the results from this brief analysis show that the bottom chord exceeds the maximum allowable stress in tension by nearly 8,000 psi, and the upper chords hold almost exactly the maximum allowable compressive stress (all values exceeding stresses that meet a safety factor of four are marked with red for compression or blue for tension) (Table 4.2).

The analysis of trusses A and E shows similar results (Table 4.1). In this case, only the bottom chord in tension exceeds the yield strength, while the rest of the elements are within a safe range. For these trusses, the symmetric verticals of each side are theoretically zero-force members.

<table>
<thead>
<tr>
<th>Element</th>
<th>Cross-Section (in(^2))</th>
<th>Force (lbs)</th>
<th>Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>1.88</td>
<td>12,950</td>
<td>6,890</td>
</tr>
<tr>
<td>B2</td>
<td>1.88</td>
<td>12,400</td>
<td>6,600</td>
</tr>
<tr>
<td>E1</td>
<td>1</td>
<td>12,160</td>
<td>12,160</td>
</tr>
<tr>
<td>E3</td>
<td>1</td>
<td>17,090</td>
<td>17,090</td>
</tr>
<tr>
<td>1-2</td>
<td>1</td>
<td>1,040</td>
<td>1,040</td>
</tr>
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<td>2-3</td>
<td>1.23</td>
<td>5,100</td>
<td>4,140</td>
</tr>
<tr>
<td>3-4</td>
<td>1.5</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

*Table 4.1 Graphical Analysis Results, Trusses A and E*

Table 4.2 Graphical Analysis Results, Trusses B-D

<table>
<thead>
<tr>
<th>Element</th>
<th>Cross-Section (in²)</th>
<th>Force (lbs)</th>
<th>Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>1.88</td>
<td>12,950</td>
<td>6,890</td>
</tr>
<tr>
<td>B3</td>
<td>1.88</td>
<td>9,300</td>
<td>4,960</td>
</tr>
<tr>
<td>E1</td>
<td>1</td>
<td>12,160</td>
<td>12,160</td>
</tr>
<tr>
<td>E2</td>
<td>1</td>
<td>12,160</td>
<td>12,160</td>
</tr>
<tr>
<td>1-2</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2-3</td>
<td>1.23</td>
<td>3,370</td>
<td>4,140</td>
</tr>
<tr>
<td>3-4</td>
<td>1.5</td>
<td>4,500</td>
<td>3,000</td>
</tr>
</tbody>
</table>

Graphical Analysis
Trusses A,E

Graphical Analysis
Trusses B-D

Figure 4.2 Graphical Analysis and Resulting Force Polygons
4.3 Arcade modeling of the trusses

To verify the previously acquired results, the trusses were modeled using Arcade to determine the stresses in each member (Figure 4.3). For these models, the load on each truss was divided equally into three point loads, and each truss was treated as an elastic truss. The results from these simulations show stresses within each member nearly identical to those given by graphical analysis (Tables 4.3, 4.4).

For trusses A and E, the most notable differences in value between the hand-calculated results and the simulation results are in elements 5 and 6, neither of which reaches a critical load according to either modeling method. The arcade results do, however, show a slightly higher value of the stress in member 1, indicating that it has a lower safety factor, although does still not carry a critical buckling load.

Arcade results for trusses B through D also produce nearly identical results to the graphical analysis. In this case, as with the other trusses, the largest difference is seen in elements that never carry a critical load.
Figure 4.3 Arcade simulation results displaying compression and tension

### Table 4.3 Arcade modeling of trusses A and E

<table>
<thead>
<tr>
<th>Element</th>
<th>Cross-Section (in²)</th>
<th>Stress, Graphical Analysis (psi)</th>
<th>Stress, Arcade Ideal Truss (psi)</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
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<td>1.88</td>
<td>6,890</td>
<td>7,150</td>
<td>4%</td>
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<td>2</td>
<td>1.88</td>
<td>6,600</td>
<td>6,690</td>
<td>1%</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>12,160</td>
<td>12,500</td>
<td>3%</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>17,090</td>
<td>16,100</td>
<td>-6%</td>
</tr>
<tr>
<td>5</td>
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<td>4,140</td>
<td>3,170</td>
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<tr>
<td>7</td>
<td>1.5</td>
<td>0</td>
<td>~0</td>
<td>0%</td>
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### Table 4.4 Arcade modeling of trusses B-D

<table>
<thead>
<tr>
<th>Element</th>
<th>Cross-Section (in²)</th>
<th>Stress, Graphical Analysis (psi)</th>
<th>Stress, Arcade Ideal Truss (psi)</th>
<th>% Difference</th>
</tr>
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<td>6,880</td>
<td>6,720</td>
<td>-2%</td>
</tr>
<tr>
<td>2</td>
<td>1.88</td>
<td>4,960</td>
<td>4,890</td>
<td>-1%</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>12,160</td>
<td>12,500</td>
<td>3%</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>12,160</td>
<td>12,500</td>
<td>3%</td>
</tr>
<tr>
<td>5</td>
<td>1</td>
<td>0</td>
<td>~0</td>
<td>0%</td>
</tr>
<tr>
<td>6</td>
<td>1.23</td>
<td>3,370</td>
<td>3,140</td>
<td>-7%</td>
</tr>
<tr>
<td>7</td>
<td>1.5</td>
<td>3,000</td>
<td>2,520</td>
<td>-16%</td>
</tr>
</tbody>
</table>
4.4 Arcade modeling of “As-Built” trusses

Due to irregularities associated with the construction of the roof, the Arcade simulations previously described represent an over-simplification of the trusses’ actual behavior. Consequently, the Arcade models of each truss were altered to more closely recreate the behavior of the roof structure as it was constructed.

For trusses A and E, there were two major changes made to the Arcade model to more closely represent the actual structure. Firstly, the diagonal members of the truss are constructed to join the upper edge of the bottom chord and the bottom edge of the upper chord, rather than receiving load directly as previously modeled (Figure 4.4). Secondly, the purlin is riveted to the upper chord almost a full five inches along the upper chord from the truss’s joint, not on the node as previously modeled.

Similar modifications were made for trusses B-D. The diagonals were altered to rest between the upper and lower chords. Also, the load from the purlin was made to rest on both the upper chord and the vertical. This change is the most significant, as the vertical that is now taking load directly from the purlin was previously a zero force element.

The results from Arcade simulations show a dramatic impact due to these changes. For trusses A and E, the upper chords are now shown to be
carrying a critical compressive load, while the rest of the stresses remain fairly consistent with previous modeling (Table 4.5). For trusses B-D, however, significant differences are seen, most importantly in the vertical elements (Table 4.6). These elements, which should theoretically carry no load, are here shown to carry a critical buckling load (Appendix A). This result is confirmed by the fact that these members visibly appear to have undergone a buckling deformation; the bowing shape is not illustrated in the 1903 drawing, and therefore was likely not part of the original design.

Figure 4.4 Arcade models of “as-built” trusses
### Table 4.5 Arcade “as-built” results, Trusses A and E

<table>
<thead>
<tr>
<th>Element</th>
<th>Cross-Section (in²)</th>
<th>Stress, Arcade Ideal Truss (psi)</th>
<th>Arcade “As-Built” (psi)</th>
</tr>
</thead>
<tbody>
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<td>1</td>
<td>1.88</td>
<td>6,720</td>
<td>7,610</td>
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</tr>
<tr>
<td>3</td>
<td>1</td>
<td>12,500</td>
<td>13,300</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>12,500</td>
<td>17,700</td>
</tr>
<tr>
<td>5</td>
<td>1</td>
<td>~0</td>
<td>1,930</td>
</tr>
<tr>
<td>6</td>
<td>1.23</td>
<td>3,140</td>
<td>4,020</td>
</tr>
<tr>
<td>7</td>
<td>1.5</td>
<td>2,520</td>
<td>~0</td>
</tr>
</tbody>
</table>

### Table 4.6 Arcade “as-built” results, Trusses B-D

<table>
<thead>
<tr>
<th>Element</th>
<th>Cross-Section (in²)</th>
<th>Stress, Arcade Ideal Truss (psi)</th>
<th>Arcade “As-Built” (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.88</td>
<td>6,720</td>
<td>6,110</td>
</tr>
<tr>
<td>2</td>
<td>1.88</td>
<td>4,890</td>
<td>4,480</td>
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<tr>
<td>3</td>
<td>1</td>
<td>12,500</td>
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<td>4</td>
<td>1</td>
<td>12,500</td>
<td>12,000</td>
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<tr>
<td>5</td>
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<td>~750</td>
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<td>1.23</td>
<td>3,140</td>
<td>3,110</td>
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<tr>
<td>7</td>
<td>1.5</td>
<td>2,520</td>
<td>1,900</td>
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</tbody>
</table>

### 4.5 Discussion

The graphical analysis and series of Arcade simulations indicate that the original truss structure was sufficient to support the self-weight of the roof and any expected live loads that may have been applied. Aside from the vertical members that failed due to buckling, every element in the truss meets at least a safety factor of three. This conclusion indicates that the
additions to the roof including iron tie rods and wooden beams were unnecessary given the current loading situation, leading to the question of why they were installed and what additional loads the roof might have been carrying during the years of the gatehouse’s use.

The effect of the five iron tie rods added to the roof depends strongly on how the trusses functioned in their as-built state. The wall plate atop the perimeter of the masonry walls seems not to be well-attached to the tops of the walls at this point, indicating that the walls are not taking any significant amount of the horizontal load exerted by the trusses (Figure 4.5). This means that under all circumstances, the bottom chords of the trusses are taking the highest forces of any member in the trusses. It is possible that tie beams were added simply to reinforce the trusses; however their presence seems excessive simply to act as a supplementary structure to a nearly stable system. The addition of these circular beams effectively doubled the volume of material in tension, thereby dramatically reducing the stress in the lower chords of the trusses. It is more likely that these tie rods were added to support additional live loads, especially given the alignment of the large crane rod hanging below the ceiling with one of these rods (Figure 4.6).

While the 1903 drawing has a note stating “the compressive stress is evidently carried mainly by 3”x4” roof timbers which are beside the
rafters,\textsuperscript{27} this in fact seems unlikely to have been the case at that time considering the placement and method of attaching the wood to the corresponding iron members. The wooden timbers are bolted to the iron rafters along their length and also rest on the wall plate running around the perimeter of the masonry walls (Figure 4.7). Assuming the wood and iron members made equal contact with the roof, the iron would, according to Arcade simulations and due to the higher stiffness of the material, take compressive load an order of magnitude higher than the wood, thereby having very little effect on the stresses in the iron. However, inspection of the roof as it stands now shows that in several places, the wood makes contact with the roof where the iron trusses do not. In this case, the wooden rafters would in fact be carrying more of the compressive load of the roof. Since it is unknown if the wooden rafters originally lay flush with the iron rafters against the roof or were as they are now, their original role in the structure cannot be determined. Additionally, due to the irregularity of their placement with regard to the roof surface now, it is unclear how much compressive load they currently take.

The full impact of the addition of iron tie rods and wood rafters to the truss system remains unclear. The fact that these additions increases the

\textsuperscript{27} Drawing by W.W. Patch. 1903
strength of the truss system to greatly exceed the values necessary to make it self-supporting, however, strongly indicates that the roof was at some point meant to carry additional loads. Also, the hooks and crane rod currently attached to the wrought iron structure were likely used at one point in the gatehouse’s history to carry some type of load. It is believed by Cochituate Park employees that the roof may have once been used to store and launch the boats that were used for inspecting the aqueduct. The roof after the addition of tie rods and wooden beams would indeed have been strong enough to support a small boat, or to hold other items related to maintenance of the aqueduct, although there is no written documentation to verify that this was ever the case.

In addition to revealing the likely uses of the roof over time as a load-bearing element, the structural evaluation also emphasizes the craftsman-like characteristics of the structural design. For example, the calculations show that the diagonal members of each truss are in compression. This is typical of traditional wood truss construction, but does not make as much sense for a wrought iron truss, as wrought iron was unique during the 1840s for its tensile strength but not cost efficient in compression compared to cast iron. Also, the vertical elements in the triangular trusses have an extremely small cross-sectional area, and could not theoretically carry any significant force in
compression; however, the method by which they are attached to the purlin on either side puts them beyond their critical buckling load, indicating that whoever constructed the roof did not have a thorough understanding of the mechanics of iron trusses (Figure 4.8).

Figure 4.5 Wall plate on masonry wall, Photograph by Dennis De Witt

Figure 4.6 Crane rod and hooks
Figure 4.7 Sistered Wooden Beams

Figure 4.8 Buckling deformation
Chapter 5: Preservation Considerations

5.1 Current conditions of the gatehouse and roof

The wrought iron structure of the roof appears to stand in functioning condition. All of the wrought iron members are coated in red paint that was apparently applied before the current wooden beams were bolted on. However, the roof and ceiling are in a deteriorated state. The roof has a large hole on the northern side, which has led to a corresponding section of the wooden bead board ceiling that has rotted away from weather damage. Additionally, there are unattached pieces of wood present in the roof structure, apparently for no purpose.

The gatehouse walls are also in poor condition; many of the bricks on the interior are hazardously loose, and on the northern wall, a layer of brick has been removed, possibly compromising the wall’s structural stability. This wall also at one point had a hole the size of a window cut out of it, although this has been filled in with brick. The windows are also boarded up, and there is graffiti on many of the building’s surfaces, including plaques installed to commemorate the aqueduct.
5.2 Ensuring structural stability and safety

The first step in ensuring the stability of the gatehouse roof would be a thorough inspection of the wrought iron trusses and rafters. A close physical inspection would reveal any cracks or large deformations in the trusses that could undermine the structure’s load-bearing capacity. A non-destructive test of the wrought iron in the trusses would also be helpful in determining the composition and quality of the material, providing a better idea of the material’s capacity in compressive strength and ductility.

For the purpose of maintaining structural safety during restoration and for the long-term, a supplementary support system may need to be installed. This would take whatever excess load is being carried by the wooden beams that must be removed to conduct a thorough inspection and preservation.

5.3 Preservation Recommendations

The final goals of a preservation project for the gatehouse will depend upon what portions of the roof are considered historic. Clearly, the trusses must be prioritized as the most historically significant part of the gatehouse in the context of American engineering; however, other parts of the roof structure are also important to the history of the building. After the wrought iron structure, the 1915 bead board ceiling is most in need of preservation.
efforts. The pattern of the boards is evidence of the care that was once taken in its design and construction. While the wooden beams were in place at least since 1903, they have since been replaced, and, as the roof will not need to bear additional loads in the future, are no longer structurally necessary. The iron tie beams may similarly be deemed unnecessary, and bear little historic significance in comparison to the trusses.

The first step that must be taken in the preservation of the gatehouse is to put at least a temporary covering over the roof. This measure will ensure that no further damage is done to the trusses, ceiling, and interior prior to a more complete restoration effort.

After the trusses are protected from weather damage, a more thorough investigation and cleaning of the wrought iron should be performed. In addition to structural characteristics, the iron should also be tested and inspected in an effort to detect any sign of paint or attachments that would provide insight into the trusses’ original state.

A long-term restoration of the building should leave the trusses visually accessible in their original context and protected from atmospheric conditions. The roof needs to be repaired or altogether replaced to protect the interior of the building. The ceiling either needs to be repaired by removing the water-damaged portions to allow visitors to view the trusses,
or the ceiling should be removed. Finally, the wrought iron needs to be restored to its original condition and configuration. Any remaining supplementary structures should be painted or constructed of a clearly different material to indicate their state as a contemporary addition to the historic structure.
Chapter 6: Conclusions and Future Work

Further archival research and physical investigation should be carried out to create a more complete history of the Cochituate gatehouse. The designer of the trusses, original drawings or payment details, and a more specific description of its uses for storage remain to be discovered.

The company responsible for the masonry work on the gatehouse and the major iron suppliers for the aqueduct, especially those in the area such as the South Boston Iron Company, could be researched more carefully as possible sources of information. It seems likely that the roof was designed by a local craftsman who may have been employed by one of these companies; thus, any of remaining documents or contracts held in these companies’ archives could contain information about the builder as well as original drawings or specifications for the trusses. Also, while Jervis’ papers have been searched for information regarding the aqueduct, Chesbrough’s personal documents, currently in the Boston Public Library have not been as thoroughly explored and may contain valuable information regarding details of the gatehouse’s engineering and construction.

Reopening the building as a visitor’s information center for the State Park would allow access to the historic roof and provide a service to the community. The park currently serves its community’s recreational needs,
and making the gatehouse accessible could open more space for use by park-goers, as well as providing a setting in which to commemorate and educate the public about the Cochituate aqueduct.

As one of the oldest standing wrought iron truss roofs in the country, the Cochituate gatehouse should be restored and opened to the public. It is significant for its place in the history of structural engineering as possibly the oldest standing wrought iron roof truss in the country, as well as for its role in one of the earliest systems to bring a public supply of water to an urban population.
Appendix A – Critical Buckling Loads

Trusses A and E

<table>
<thead>
<tr>
<th>Element</th>
<th>Cross-Section Geometry</th>
<th>Cross-Section Area (in²)</th>
<th>Length (in)</th>
<th>Min.* (lbs)</th>
<th>Max.** (lbs)</th>
</tr>
</thead>
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<td>45,300</td>
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Trusses B-D

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<th>Cross-Section Geometry</th>
<th>Cross-Section Area (in²)</th>
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<td>598</td>
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*Critical buckling load assuming single-support boundary conditions
**Critical buckling load assuming double-clamped boundary conditions
***These are double chord members. Two identical members carry the total force as shown in the calculations previous described.
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