History and Technology of Lemuel Chenoweth's Covered Bridges

by

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ABSTRACT

Lemuel Chenoweth was a carpenter and bridge builder who played a key role in the development of the infrastructure of antebellum Virginia. Theodore Burr and Lewis Wernwag are among the designers who influenced the structure and construction of his bridges, two of which are currently standing in West Virginia. The timber covered bridge at Beverly is one of Chenoweth's key creations that have been lost, which was at the time located on a key turnpike running through the county seat.

The first goal of the following study is to establish the geometry of the Beverly Bridge. To do this, historical photographs of the construction and the finished bridge were studied. Salvaged timbers from the bridge were observed to establish the cross-sectional dimensions and species of the wood. Finally, surveys of Chenoweth's existing bridges were performed to determine the probable joinery and truss dimensions. A second goal is to perform a simplified and finite element analysis of the bridge in order to determine its performance under modern vehicle loading.

A third goal is to determine the feasibility of reconstruction of the bridge. The Beverly Bridge is compared to other existing timber covered bridges of a similar span and type in order to prove that similar bridges can withstand modern loads adequately. Modifications that may be made to the bridge are then discussed, covering both structural and nonstructural considerations. Finally, the cost of reconstructing the bridge today is assessed.

Thesis Supervisor: John Ochsendorf
Title: Professor of Civil and Environmental Engineering and Architecture

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First, I want to thank John Ochsendorf for introducing me to this amazing topic and guiding me along the way. I have gained a huge appreciation for the history of construction while working on this project, and it could not have happened if I had worked with anyone else. Speaking with him has permanently altered the way I view structures and has opened my eyes to the beauty of many of the buildings surrounding me.

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A final recognition goes to my colleagues in the Structural Design Lab and my M.Eng. classmates for the incredible feedback and questions along the course of the project. Working beside you all has made me a much better engineer, and listening to you speak about your projects is continually inspiring.

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1 History and Context of Lemuel and Eli Chenoweth

The following chapter will serve as an introduction to the Chenoweth brothers, who built over a dozen timber covered bridges during the 19th century. There will be a brief overview of the development of timber covered bridges in America in order to provide context for the structures built by the Chenoweths. The history and construction of their significant structures are discussed. Finally, the history of their covered bridge at Beverly is discussed in more detail, including its demolition in the 1950s.

1.1 Development and Characteristics of Timber Covered Bridges

During the first half of the 19th century, a tremendous growth in commerce in America created a demand for bridges on a burgeoning system of turnpikes and railways. The development of timber bridges has been well chronicled by engineers and historians; and can be traced by the progression of bridge patents during the early 1800s. Timber became the material of choice for many of the longest span bridges, and an abundance and variety of types of timber bridges emerged. After several failures due to deteriorated bridges, the structural timbers of new bridges were protected from the weather and moisture with a sacrificial and removable roof and siding. The first timber bridge constructed with a roof and siding was the "Permanent Bridge" over the Schulkill River, built by Timothy Palmer (1751-1821) and completed in 1805 (Allen 1959).

The first major innovation was the combined arch and truss model, patented by Theodore Burr in 1817 (Kemp 2005). This design allowed for longer spans than standard truss designs while creating a level roadway. Steven Harriman Long modified the Burr system by using two cross-bracing diagonals in each truss panel, as opposed to the single compression diagonal member used in Burr's designs. As iron become more readily available, the use of all-timber bridges steadily declined. William Howe patented his bridge system in 1840 that made use of a threaded iron rod in place of the vertical truss elements. This innovation allowed for simplified construction and adjustments to the bridge as it deforms over time.

This means that many bridges were built by local carpenters and according to local rules of thumb (Pierce 1999). It is estimated that there were as many as 10,000 timber bridges in the United States by 1885 (Ibid.). As of 2009, there were 822 known timber covered bridges worldwide, most of them in rural areas (Wright 2009). Many of the existing bridges continue to carry vehicle traffic, and there are a few timber railroad bridges still in existence.

A common concern associated with timber bridges is their susceptibility to floods. Timber's low density means that hydrostatic forces would be a concern under high water conditions. The Blenheim Bridge, which at 210 feet was the longest span covered bridge in America, was washed away during Hurricane Irene in 2011 (Eckholm 2011). Another concern is that over time, rot and degradation may reduce the capacity of the timbers if the cover leaks due to lack of maintenance or if roadway drainage can flow unrestricted onto the structure (Pierce et al. 2005).

Many covered bridges today feature "running planks," which are heavy timber boards that are sacrificial and rest on top of the timber decks. The boards guide traffic along the center of the bridge, allowing the vehicle loads to be transferred evenly to the exterior trusses.

The use of timber bridges in new construction had heavily declined during the early 20th century and nearly ceased by the 1930s (Pierce et al. 2005). Some were demolished because they could not support the loads of newer trucks; others because the trusses did not have adequate clearance. Some were destroyed by fire or flooding. In recent years, there has been a growth of interest in constructing new timber covered bridges in America. Of the thirty states that currently contain a timber covered bridge, more than half have built a new one since 1975. This renewed passion for the craft could spur a push to reclaim and rebuild significant bridges that have been lost over time.

1.2 Lemuel and Eli Chenoweth

Lemuel Chenoweth was born in 1811 and became a designer and builder of timber bridges during the mid 19th century. He originally worked as a carpenter and furniture maker after moving to the town of Beverly, in what is now West Virginia, in 1835. Chenoweth followed in the footsteps of famed bridge designer Lewis Wernwag, who was born in Germany in 1769. Wernwag is best known for building what is known as "The Colossus," a 340 foot span timber arch bridge over the Schuylkill River in Pennsylvania, which opened in 1813. This bridge was the longest span in the Americas until it burned down in 1838 (Griggs 2004). Wernwag also owned a metal works company and was a pioneer in combining timber and iron for use in his long span bridges (Pierce et al. 2005). He developed what he called the "Economy Bridge," which was constructed primarily with timber but sometimes included iron bracing elements. They featured a single iron through-bolt allowing for ease of construction and maintenance at each timber connection. This feature is present at both of Chenoweth's standing bridges.

In 1831, the state of Virginia set out to build a turnpike following the success of new federal and state highways. The main bridges for this road were designed by Wernwag but built by his sons and their foremen (Allen 1959). One of the last bridges designed by Wernwag was over the Cheat River in what is now Preston County, West Virginia. The bridge was built in 1834 as a part of the Northwestern Virginia Turnpike and was destroyed by fire in 1964 (Auvil 1973). The two span, 339-foot bridge was in Preston County and is roughly 50 miles away from the Chenoweths' hometown of Beverly. It is constructed after the Burr truss plan by Josiah Kidwell. Wernwag died in 1843, but his sons and construction foremen continued in the bridge building business. Other bridge builders constructed many of Wernwag's designs, so it is unclear how many projects he finally influenced.

The bridge building career of the Chenoweths began when the Staunton and Parkersburg (S&P) Turnpike was approved. The turnpike was chartered in 1817, and construction began in 1831. The turnpike completed a continuous road network from Richmond, Virginia to the Ohio River, which at the time included what is presently West Virginia, greatly improving the means of trade within the state (Sturm 2010). By 1845, road construction had finished, allowing for the beginning of construction of the bridges. One of the major bridges to be constructed as a part of the S&P Turnpike was near Lemuel Chenoweth's residence, at Beverly, Virginia. The bridge was to span over the Tygart's Valley River. Wernwag is credited as the designer of the bridge (Auvil 1973, Carmody 1941), but Chenoweth was awarded the contract for the construction of the bridge superstructure. Wernwag had passed away prior to the award of the contract, but it is possible that another designer in his company supervised Chenoweth in the design and construction of the bridge. Along with his brother Eli (1825-1895), Lemuel Chenoweth (1811-1887) went on to build up to 15 timber bridges throughout what is now West Virginia (Kemp 1984).

The bridges built by the Chenoweths have shown an impressive longevity and strength. His bridges at Barrackville and Philippi are currently standing, which is a notable achievement of serviceability since vehicle loads increased significantly over their lifetimes. The Barrackville Bridge was originally constructed in 1853 as a part of the Fairmont-Wheeling turnpike (Kemp

1975). It spans 145’ 5” over Buffalo Creek and is a multiple King-post arch-truss (Figure 1.1), the system patented by Theodore Burr in 1817. The Historic American Engineering Record (HAER) conducted an extensive survey of the bridge in 1973, including photographs and measured drawings (HAER WV-8 2003). The survey shows the modifications made to the bridge in order to carry 20th century vehicle loads. The deck framing was strengthened with additional timber stringers. Iron rods were added that hang the roadway from the deck, though this has been shown to add redundancy to the structure, rather than strengthen it (Lamar and Schafer 2002). The bridge carried vehicle traffic on Marion County Rt. 21 until 1991, when a modern steel girder bridge replaced it. In 1999, the bridge was renovated so as to match its original appearance and currently serves only pedestrian traffic.

Figure 1.1: Barrackville Covered Bridge (photograph by author, 2014)

The Philippi Bridge is the most well known Chenoweth structure, and is the only one that currently services vehicle traffic (Figure 1.2). It was opened in 1852 and originally consisted of two 138-foot spans that meet at a masonry pier in the Tygart River in the city of Philippi (Kemp 1992). The structure had two parallel carriageways and featured two crossing diagonal members at each truss panel. The bridge was the site of the first Civil War land battle in 1861. A major renovation of the bridge was undertaken in 1938, when a concrete pier was added in the middle of each clear span in order to support steel beams and a new concrete deck needed to support highway loadings. This modern deck carries the full live load, and the original timber supports only itself and the cover. The bridge was nearly destroyed by an accidental fire in 1989 caused by a gasoline spill from a nearby storage tank. The bridge was restored by 1991 and retained as many of the original timbers as possible. It currently services U.S. Route 250 and is the only timber bridge that is a part of a federal highway (Ibid.).

Figure 1.2: Philippi Covered Bridge (photograph by author, 2014)

The Chenoweths had a meaningful impact on other bridges built in western Virginia during the mid 19th century. The Carrolton Bridge in Buckhannon is a 155 foot span Burr Arch bridge built by the O'Brien brothers, who served as the masonry contractors on Chenoweth's earlier projects (Kemp 1984). This bridge was completed in 1856 and was rebuilt in 2002. It is currently the longest all timber bridge currently carrying traffic in West Virginia. Philippi, Barrackville, and Carrolton hold the distinction of being the three oldest and longest covered bridges that survive in West Virginia today. All are classic examples of the Burr Truss system and have served a major role in the history and development of the region, from the years prior to the Civil War through the present day. The locations, construction dates, and spans of all known bridges built by the Chenoweths are given in Appendix A.

1.3 History of the Beverly Bridge

Originally built in 1847 and rebuilt in 1873, the covered bridge at Beverly holds the distinction of being both the first and last bridge built by Lemuel Chenoweth (Allan 2006). The Staunton-Parkersburg Turnpike was constructed beginning at its two ends, and the roadway was completed when the two met in the middle at Beverly, which was the seat of Randolph County at the time. The superstructure was contracted to Lemuel and Eli Chenoweth, and the stone abutments constructed by Daniel Kalar. Soon, Lemuel constructed a house in Beverly directly adjacent to the entrance to the bridge. During the Civil War, the Beverly Bridge was burned down, and Chenoweth built a replacement bridge on the same abutments in 1873 (Chambers

2004). According to an article in *The Elkins Intermountain*, a local newspaper, the bridge was "in a good state of preservation" and was one of five bridges built by Chenoweth still in use in October 1950. Soon after, the bridge was slated for demolition. A common reason given for the demolition was that the bridge did not have enough clearance to accommodate school buses. There was an active push within the community to maintain the bridge, especially from the Beverly Historical Society. According to the *Intermountain*, "it was felt that immediate action was necessary if {the bridge} is to be saved" (May 21 1951), which suggests that local citizens were opposed to the destruction. However, this push was unsuccessful, as the bridge was demolished in 1951.

The story of the demolition of the bridge demonstrates that the engineers involved may not have accurately known the strength of the structure. According to Mr. Allan, the bridge was slated for demolition in the late fall. Engineers placed dynamite at the abutments of the bridge. When the charge was ignited, the bridge remained in its position. A second charge of explosives was prepared, which was only able to remove one end of the bridge from its abutment. Finally, a bulldozer was rigged up to the remaining structure, and it was torn free of its abutment and into the Tygarts Valley River.

The Beverly Bridge was a major loss to history and engineering knowledge, as it was one of the major works of one of the most prolific and talented timber bridge builders of the 19th century. The bridges have shown tremendous workmanship and longevity, as two are still standing over 150 years after they were constructed. They had also shown the ability to carry a higher load than expected at the time of construction with minimal modern changes to the structure. If the original bridge were standing today, it would be the oldest and would be tied for the longest span for a covered bridge in West Virginia (Wright 2009). It represents the technological peak of the Chenoweths, as it was their longest single span and the final bridge they constructed. Their bridges at Marlinton and Philippi are their only structures with greater overall lengths, but both consist of two separate spans.

1.4 Problem Statement

The goal of this thesis is to determine the physical dimensions of the now demolished Beverly Covered Bridge and evaluate the feasibility of reconstruction of the structure, including an analysis of its performance under modern vehicle loads. The first step is to determine the physical characteristics of the bridge, including materials and member sizes. Following that, a series of analyses are shown for different assumptions of the load path through the structure. An attempt is made to determine the maximum uniform load that can be carried by the structure and to determine its ability to handle modern traffic loading. These results are then compared with that of a finite element model.

The second chapter of this thesis will focus on the original Beverly Bridge. Records and historical photographs are studied in order to determine the general dimensions and structural systems used by the bridge. Chenoweth's other standing bridges and salvaged timbers from the Beverly Bridge are also examined in order to establish dimensions of the members and connection details.

Chapter three is the analysis of the bridge. Simplified methods of analysis are compared to a finite element model of the bridge trusses. Wind loading and dynamic properties of the bridge are also studied.

The fourth chapter examines the feasibility of reconstructing the bridge. The cost of reconstruction is estimated. Potential structural and non-structural modifications to the bridge are also discussed. The final goal of this thesis is to show how the Beverly Covered Bridge could be rebuilt today.

2 Investigation into the Beverly Bridge

The first step in rebuilding the Beverly Covered Bridge is determining its physical characteristics. To do this, historical photographs were studied to determine the overall truss geometry and framing scheme. Randy Allan, a forester and the owner of the Lemuel Chenoweth House and Museum, was interviewed about the contracts and records of the bridge (personal conversation with Allan). The existing bridge abutments were studied, and surveys were taken of the two surviving Chenoweth bridges. Finally, drawings and photographs of the Barrackville Bridge from a study conducted by the Historic American Engineering Record (HAER) in 1973 were studied in order to determine the modifications made to the bridge for it to carry modern traffic. The goal of this chapter is to establish the original dimensions and construction details of the Beverly Bridge.

2.1 Beverly Bridge Documentation

Allan performed research into the construction contracts won by the Chenoweths at the Library of Virginia in Richmond. The information was published in his book, *Bridging the Gaps* (Allan 2006). The contracts called for a single lane bridge over the Tygarts River. Chenoweth won the contract to construct the timber superstructure, though there do not appear to be any other bidders. A fee was paid in order to use the combined king post and arch system, which at the time was still protected under a patent by Theodore Burr.

Observations were performed by the author in January 2014 of the current bridge at Beverly and the two extant bridges built by Chenoweth. At Beverly, the stone abutments are still present and are used to support the modern bridge (Figure 2.1). Concrete was added above the stone to extend the face of the abutments. This makes it impossible to measure the exact original span length. Concrete was also added along the sides of the abutments to allow for a two lane roadway, rather than the original 18-foot-wide passage. Intermediate concrete piers were also built in the Tygarts River. Lemuel Chenoweth's house is still standing at its original location overlooking the Beverly Bridge, and currently operates as a museum (Figure 2.2). Many buildings in the town have been maintained in their historical condition, and the Beverly Historical Society is active in preserving the elements of the town.

Figure 2.1: Existing Beverly Bridge Abutment (photograph by author, 2014). The concrete abutment for the current bridge was cast around the original masonry structures.

Figure 2.2 Current Beverly Bridge Roadway, with Chenoweth's House on right (photograph by author, 2014)

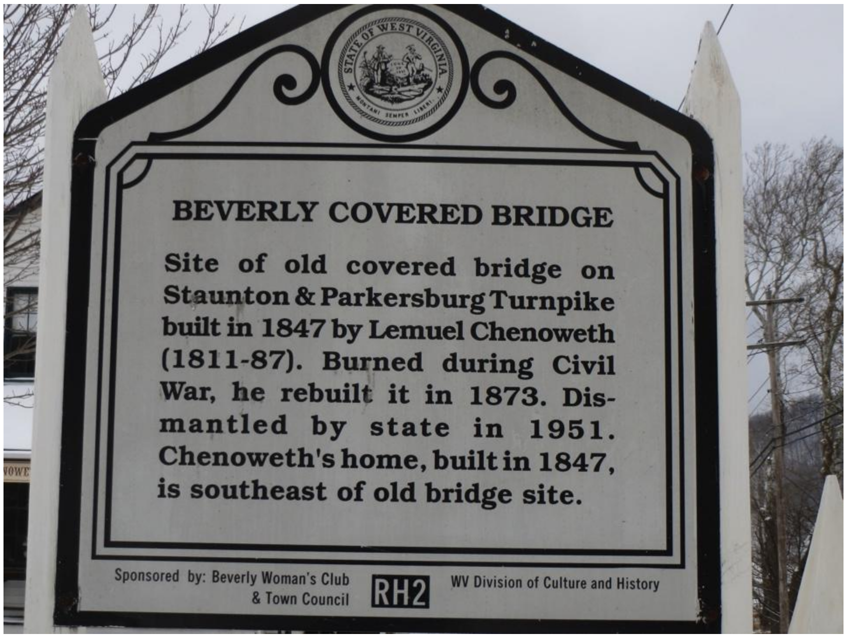

Figure 2.3: Highway Marker at Bridge Location

Historical photographs were examined in order to verify the general layout and location of the Beverly Bridge (Figure 2.4, Figure 2.5, and Figure 2.6). These show that the bridge was constructed as a Burr truss, with the diagonals in compression and cross bracing diagonals at the panel resting on the abutments. It also shows that the bridge entrance is adjacent to a railroad, as are the observed abutments. Additionally, the photographs show the bridge opening was changed from the traditional rounded shape to a square profile in order to accommodate large vehicles. The photographs show a posted load limit of three tons. It also appears that the bridge experienced significant sag over time. The photographs show that the arch crosses the bottom chord at the third panel of the truss. It can also be observed that there are sixteen truss panels, including the two that sit completely on the abutments. There are also iron bolts visible where the arch meets the diagonals and verticals of the truss. This construction makes the Beverly Bridge similar to the Barrackville, with the exception that Beverly has one additional truss panel. It is not clear if the bridge was cambered originally, and no exact date of the photographs is available.

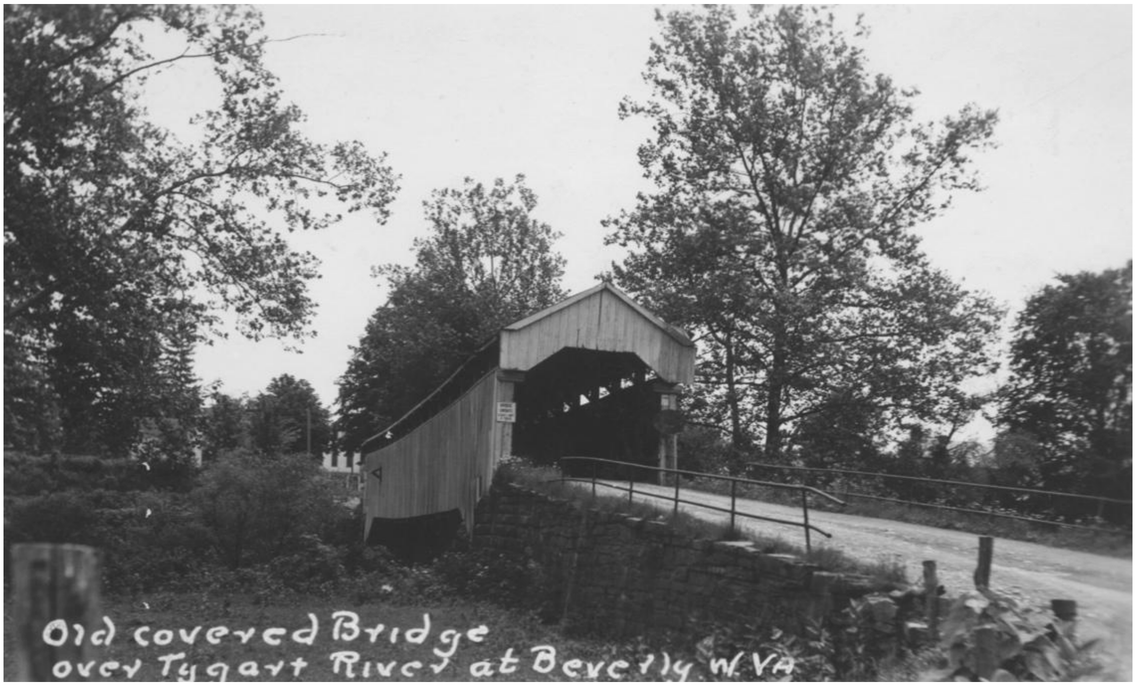

Figure 2.4: Beverly Bridge Photograph (Source: Ochsendorf Personal Collection)

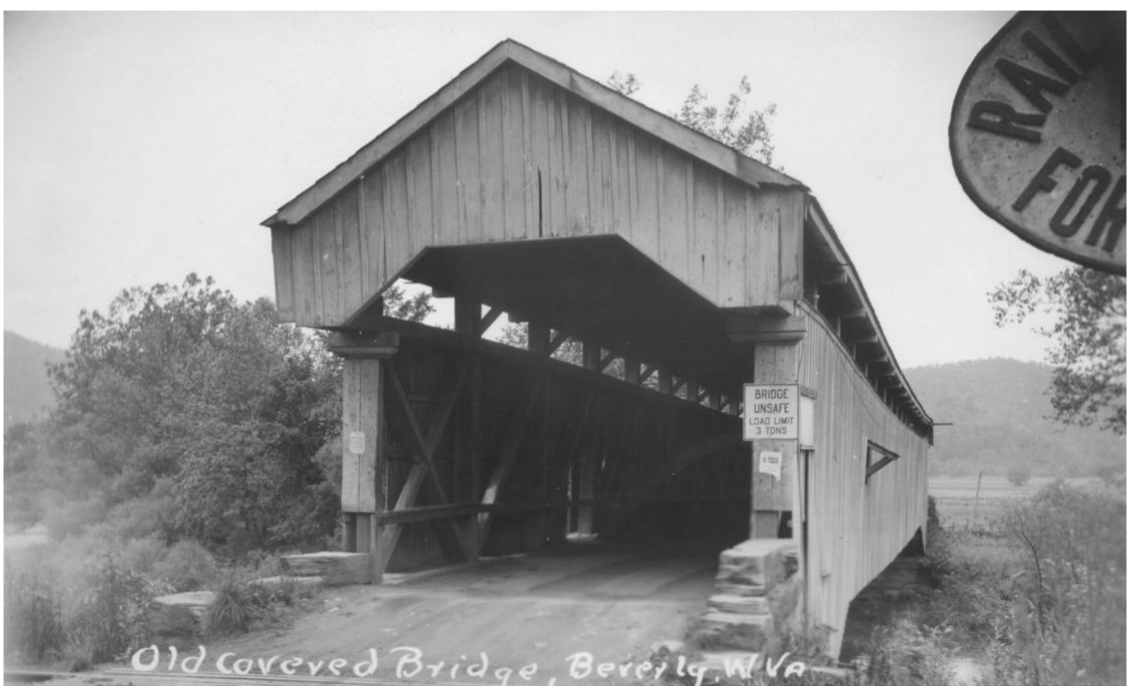

Figure 2.5: Beverly Bridge Photograph (Source: Ochsendorf Personal Collection). Detail from interior is shown below

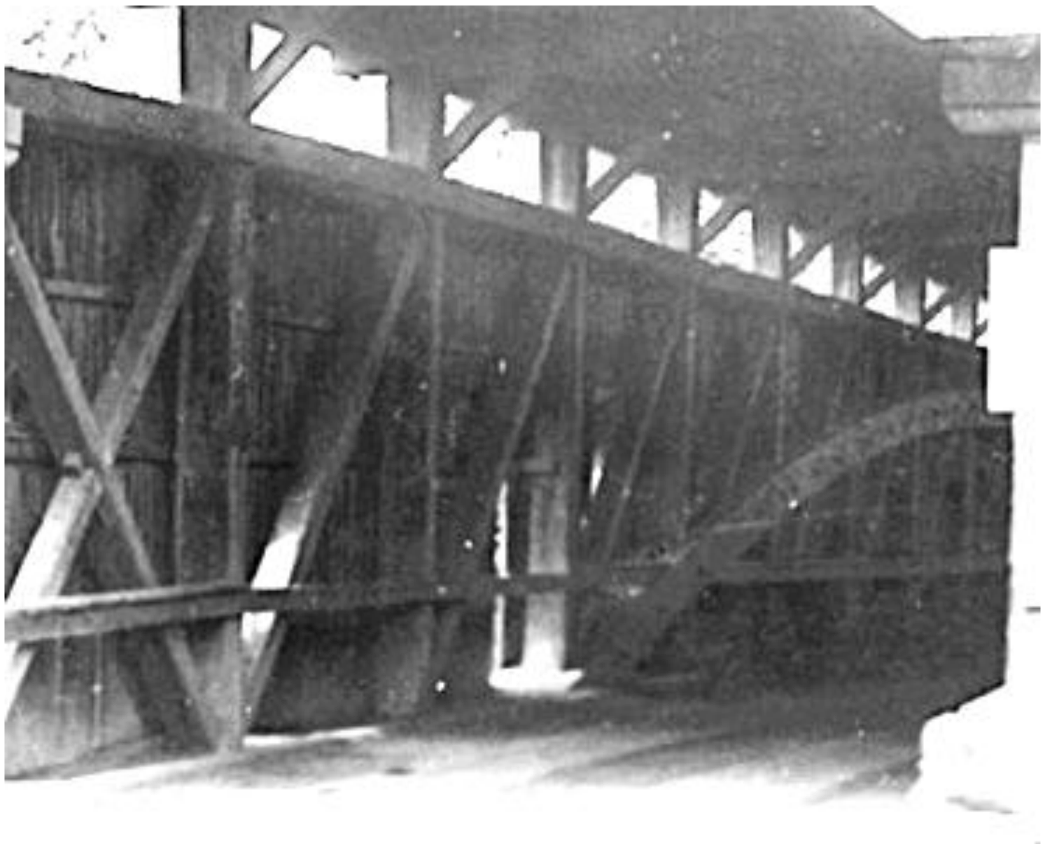

Figure 2.6: Beverly Bridge photograph detail, showing a Burr truss layout and typical connections

2.2 Beverly Bridge Geometry and Properties

After the bridge was torn down, local townspersons scavenged the timbers from the river. Joseph and Heather Biola (a member of the Beverly Historical Society) own a house at 12 Prospect Street in Elkins that was partially constructed using the salvaged timbers (Figure 2.7). The members were examined with the help of Darryl Weiser, a local carpenter who has worked on several covered bridges in Virginia. They were inspected for signs of wear and damage, as well as to gain species and age information. Cross-sectional dimensions were also measured.

Figure 2.7: Portion of Beverly timbers in Biola house (photograph by author, 2014)

The individual elements show no signs of deformation, though the historical photographs show a bridge that is clearly lower in the middle than at the abutments. The ends of the members, where the tree rings are visible, were examined in order to determine the species and gauge the age of the timbers used. The members believed to be the bottom chord and vertical members are yellow poplar, which is an easily workable material when green, meaning freshly cut when alive. Care must be taken to account for shrinkage of the timbers as they dry out, or damaged members or unwanted initial stresses may be present.

At the ends of the timbers, the rings of the timbers were exposed. This provided useful information about the properties of the timber used in the bridge. The poplar had between 25 and 30 rings per inch. This is a high value for yellow poplar, which today typically has 10 rings per inch (Forest Products Laboratory). This high density of rings is typical of old-growth forests, where close spacing of neighboring trees led to slower growth. Based on the ring density and diameter of the timbers observed, it is estimated that the trees used in the Beverly Bridge were between 225 and 270 years old. According to Mr. Weiser, all of the forests containing trees that age or older were completely cleared by the 1920s. The denser wood would have a higher strength and lower modulus of elasticity than modern timbers of the same species. However, the modern wood would have a lower dead load if the same cross sections were used. For the purposes of the analysis to follow, the structure will have the same cross sections as the original Beverly Bridge, but modern timber mechanical properties will be used.

The salvaged timber members directly supporting the Biola roof measure 16” x 9”, typically. These are likely part of the bottom chord of the truss, since there are cut out sections for both vertical and arch members. Additionally, the members are longer than a truss panel, and the chords were constructed using continuous members on Chenoweth’s other bridges. The chord members are resting on what is believed to be a vertical member of the truss, which measures 12 ¾” x 8”. This timber may be a vertical that was located near the abutments, where it would extend beyond the bottom chord of the truss. It contains cutaways that could fit the arch and the bottom chord. The vertical posts of the addition measure 8” x 8” and are between six and eight feet long. These are believed to be diagonal members of the truss sawed in two transversely. Their cross section is similar to that of the diagonals at Barrackville, and the diagonals at the Beverly Bridge would measure 16 feet in length. There were no arch or top chord timbers at the house, and cross-sectional dimensions for these members were assumed based on those at Barrackville.

Observations of the Barrackville Bridge were also performed in order to gain information about the construction details and structural properties that were not evident in the historical photographs or salvaged timbers. Barrackville is a single lane bridge, unlike the “double-barreled” Philippi. The truss panel dimensions are also very similar. This assumption is taken because the shortest span between abutments is less than two feet shorter at Beverly than at Barrackville and the abutments extend further beneath the bridge at Beverly. The bridges were likely both 18 feet wide, as was specified for all single lane bridges on the Staunton and Parkersburg turnpike. It appears from the photographs that the clear heights are also similar, so it is reasonable to assume that each Beverly truss panel is the same dimension as those at Barrackville.

The bottom chord of the Barrackville Bridge is composed of two parallel pieces, so a member size of 16” x 18”, or double the observed chord cross section, will be used in the analysis. The top chord is also composed of two separate timber pieces on either side of the vertical truss member. No examples of this piece could be located, so it will be assumed that the members are the same size as the top chord at Barrackville which are 10” x 6”.

Barrackville provides a useful point of reference as to how the floor framing of the Beverly Bridge likely changed over time. The bridge was renovated so as to match its appearance at the time of its construction in 1999, and its current floor framing is shown in Figure 2.8. There is one primary beam spanning between the trusses at each panel intersection, with a steel rod attached underneath to form a kind of shallow truss. Small secondary timber beams frame into the primary beams and truss bottom chords at roughly 45 degrees. These secondary beams support two layers of timber planks.

Figure 2.8: Barrackville Floor Framing (photograph by author, 2014)

This system differs significantly from what was documented by HAER in 1973, when the bridge carried vehicle traffic. An excerpt from a plan view drawing from the HAER survey is shown in Figure 2.9. The figure shows several floor beams spanning between the two trusses. The floor beams support perpendicular stringer beams, which in turn support the timber roadway deck. This shows that high concentrated wheel loads dominate the design of the floor system, as opposed to the lighter surface loads used in 19th century construction. Today, the floor system would have to be completely redesigned to meet vehicle bearing standards, and no documentation of the floor system used at the Beverly Bridge is available. Also of note is the lateral steel cross bracing located on the underside of the road deck at Barrackville, which was later removed. This system was adapted for use in the following analysis of Beverly (see Chapter 3) in order to reduce lateral deflections of the deck.

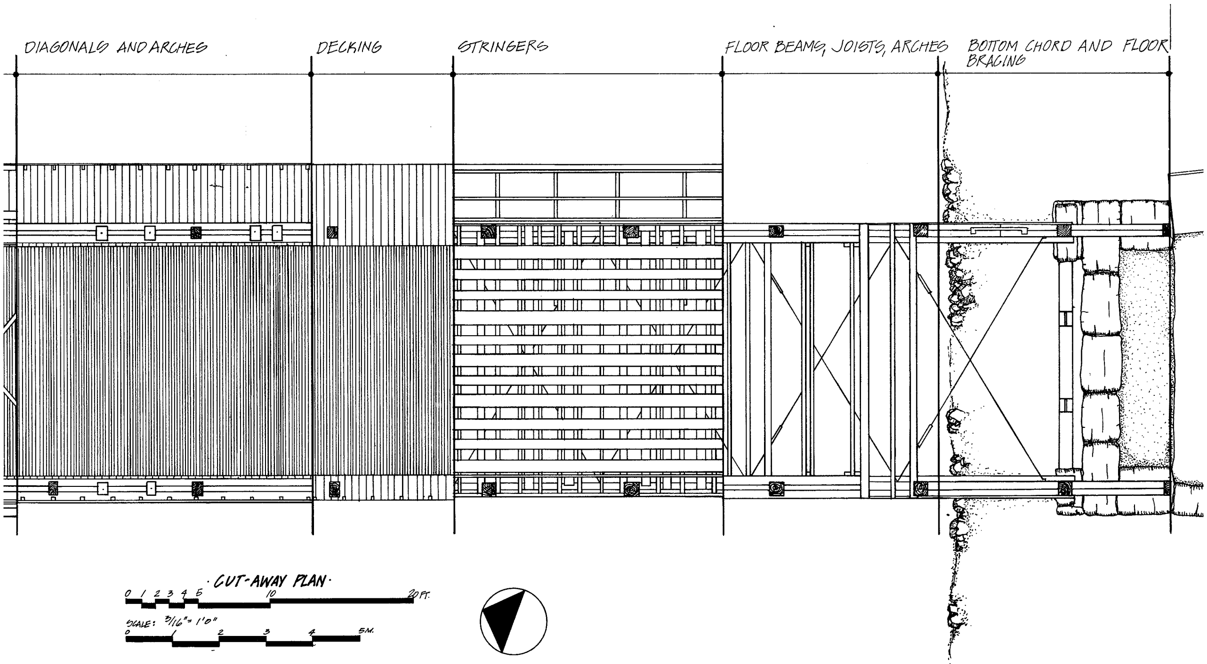

Figure 2.9: Partial Floor Plan of Barrackville Bridge (HAER, 1973)

A section and a detail of the Beverly Bridge are shown below in Figure 2.10 and Figure 2.11. For the purposes of this study, the original stone abutment span will be used. All members with the exception of the truss diagonals are composed of yellow poplar. The diagonals are assumed to be white oak. It is possible that all of the other compression elements (meaning the top chord of the truss and the arch) would be composed of oak, as well. Yellow poplar has a lower compressive strength than oak, so using yellow poplar properties in the analysis will yield a conservative estimate of the overall bridge capacity (Wheat and Cramer 2012).

The figures below represent the proposed geometry for the Beverly Covered Bridge. Dimensions are based on those measured at the Biola house and the Barrackville Bridge. The spans of the two bridges are similar, so it would be reasonable to expect that member cross sections are also similar. If additional photographs or documentation of the original condition to the Beverly Bridge could be found, then the geometry would be changed to reflect it.

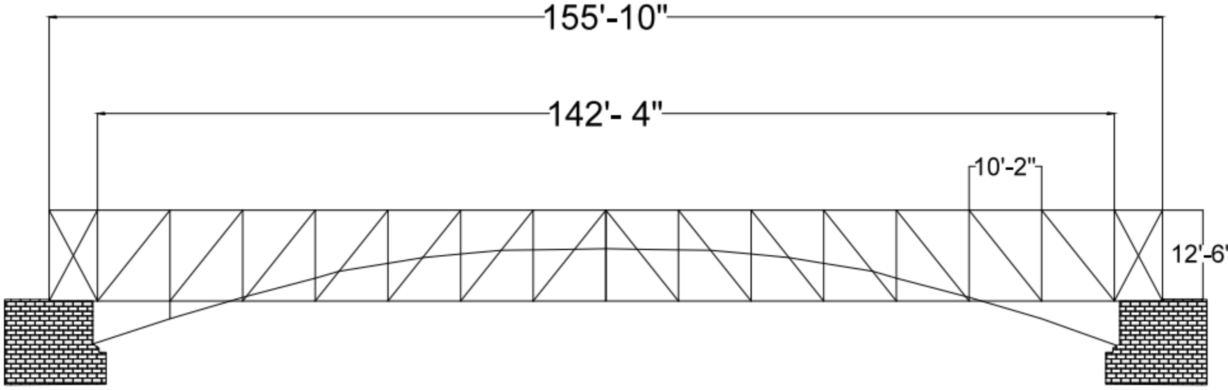

Figure 2.10: Beverly Bridge Section

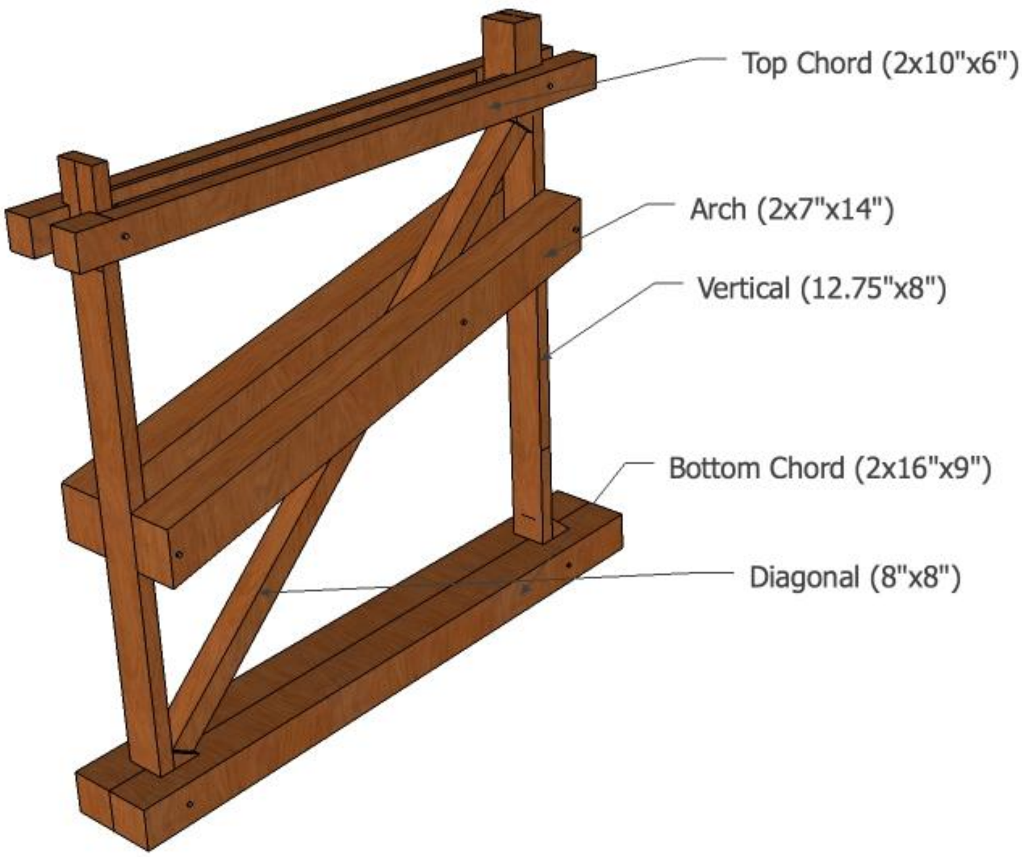

Figure 2.11: Beverly Bridge Typical Truss Panel Near Center of Span

The goal of this chapter was to establish the geometry of the Beverly Bridge. The geometry of the trusses proposed by the author is summarized in the above images. There is some uncertainty in the member sizes, and further study of salvaged timbers or historical records could improve the proposed geometries. One key finding from this study is that the material differs from what was given by historical records. The bridge was stated to be composed of poplar, but the timbers examined were both poplar and oak. It is unclear what components were composed of each species. For the purposes of this study, all elements except for the diagonals in the gravity bearing trusses are assumed to be composed of yellow poplar. The proposed geometry will be used to evaluate the bridge's structural performance in the following chapter.

3 Analysis of the Beverly Bridge

The following chapter will assess the structural performance of the Beverly Bridge, using the geometry and dimensions determined from the previous chapter. Studies of similar bridges are discussed, and the results are compared with the analysis at hand. Analysis is performed for AASHTO loadings, both for simplified models and a finite element model of the combined arch-truss. Three different models are tested under

- Self weight
- Self weight and a 15-ton moving load
- Self weight and a lane live loading

The dynamic properties and lateral loading of the bridge are also discussed.

3.1 Precedent Studies

Lemuel Chenoweth's bridges and Barrackville and Philippi have previously been analyzed for modern traffic demands. A paper by Emory Kemp and Paul Marshall documented the renovation of the Philippi Bridge after a 1989 fire (Kemp and Marshall 1992). The timber structure no longer supports vehicular loads and no analysis results are specified, but the paper confirms that Chenoweth's bridges were constructed primarily using green Poplar logs. It also notes that tensile capacity of some truss members was a concern, and many were reinforced with steel rods.

Kemp was also an author of two papers, which provide detailed analyses of Barrackville. The first, "Case Study of Burr Truss Covered Bridge," was written in 1975 and studies the bridge's performance under gravity loading (Kemp and Hall 1975). Both the truss by itself and the combined arch-truss system were analyzed with the diagonals and vertical members pinned to the chords, and all other connections fixed. The bridge was analyzed under both a 60-ton truck and a historical uniform load of 33 psf. The study accounts for a higher allowable stress for seasoned wood. The analysis led the authors to conclude that the arch carries a significant portion of the load, as truss member stresses were greatly reduced compared to the truss-only model. Deflections were reduced by a factor of four between the truss alone and the combined system. This stiffening is achieved with only a 12% increase in the total dead load of the structure. However, it is noted that long-term deflections are a more substantial concern. Over time, shrinkage of the material and loosening of the joints after years of loading can cause significant sag in the bridge.

Kemp, Spyrakos, and Venkatareddy studied the Barrackville Bridge's performance under seismic loading in 1996 (Spyrakos, Kemp, and Venkatareddy 1996). It was noted that the lower chord joints of the bridge showed significant deterioration, but a fully repaired system would accommodate AASHTO loading. It was also shown that the bridge would be severely damaged under a Mercalli VIII scale earthquake. The damage would be focused primarily on the top bracing members and the top crossing beams. The authors recommended that both seismic analysis and repairs should be performed at the same time as nonseismic repairs for historic timber bridges in order to maximize economic efficiency. Also, the members intended for wind bracing may not be able to withstand high seismic loads.

Over the past several decades, several covered bridges were surveyed and analyzed as part of the Historic American Engineering Record documentation project. The Pine Grove Bridge in Pennsylvania is the only Burr Truss bridge studied under the program (Lamar and Schafer 2004). The 90-foot span was built in 1884 was studied in 2002 for vehicular loads. The bridge follows a similar general layout to the Beverly Bridge. The truss supports the roadway and is sandwiched by two arches on either side of the bridge. The authors studied the bridge under 5-ton point loads. The truss alone was studied first, then the arch alone, and finally the combined system. Under dead load, the arch alone and the truss alone deflected 0.91 and 0.96 inches respectively. The combined system deflected by only 0.25 inches under the same load. The stiffness of the combined system was found to be almost double the sum of the individual stiffnesses.

Under a midspan point load, the arch alone had a calculated internal bending moment of 304 kip-inches, which fell to 10 kip-inches according to the combined model. It was shown that stresses and live load deflections are well within acceptable limits for the bridge. It was also shown that the arch carries three times the load of the truss, though the truss does greatly reduce bending moments in the arch.

The Federal Department of Transportation published a Covered Bridge manual in 2005, which was intended for use by practicing engineers evaluating existing bridges. For combined arch-truss systems, two simplified methods of analysis are suggested. The first method is to treat the arch as merely bracing for the truss and not as a load-bearing element. A second method proposes studying the two systems separately, with the arch carrying the live load and the truss carrying the dead load. Kemp's study of the Barrackville Bridge showed that the addition of the arch increases the vertical stiffness of the bridge by almost a factor of four. The studies by Lamar and Schafer show that both of these techniques would misjudge the capacity of the Pine Grove Bridge, since the truss acts to distribute live load to the arch, and the arch is extremely effective at carrying the uniformly distributed truss dead load. At Pine Grove, an analysis that ignores the load capacity of the arch would lead an engineer to conclude that the bridge would not be able to carry traffic. Likewise, assuming that the arch carries live load would lead an engineer to conclude that high bending moments would overstress the arch when loaded at the quarter span. Both conclusions were shown to be incorrect when combined arch-truss behavior is modeled. In the following studies, two possible simplified analyses are performed and are then compared to a combined arch and truss model.

3.2 Methodology

Analysis of the bridge was performed using the National Design Specification for Wood Construction (NDS). The supplement to the manual lists material properties for most species of timber used in American construction. The values are based on the lowest 5% of tested materials. Properties used for both white oak and yellow poplars are shown in Table 3.1.

Table 3.1: Design Material Properties

Species	Modulus of Elasticity (ksi)	Bending Strength F_b (ksi)	Tension Parallel to Grain F_t (ksi)	Compression Parallel to Grain F_c (ksi)	Density (pcf)
White Oak	1,100	1.2	0.70	1.10	44.2
Yellow Poplar	1,500	1.0	0.575	0.90	30.6

The material properties for wood can vary by quite an extreme amount. For example, the Forest Products Laboratory publishes timber structural properties, and the critical stresses are consistently higher than those used in design (see Table 3.2). Conservative estimates of strength are reasonable, since material is often not loaded directly along grain lines, and knots in the material can significantly reduce the strength.

Table 3.2: Material Properties from Forest Products Laboratory (12% moisture content)

Species	Modulus of Elasticity (ksi)	Bending Strength F_b (ksi)	Tension Parallel to Grain F_t (ksi)	Compression Parallel to Grain F_c (ksi)	Density (pcf)
White Oak	1,780	15.2	--	7.44	40.6
Yellow Poplar	1,580	10.1	--	5.54	26.2

Material stresses for combined bending and axial load were evaluated using a unity check, where the demands are compared to the capacity through Equation 3.1.

$$\frac{f_a}{F'_a} + \frac{f_b}{F'_b} \leq 1 \qquad \text{Eqn 3.1}$$

Where f_a=applied stress in tension or compression

F^1_a=adjusted tension or compression capacity

f_b=M/S (moment divided by section modulus)

F'_b=bending strength

The capacities of timber members are to be adjusted for various factors, including size, temperature effects, and applied fireproofing to the material. For members with a cross-sectional depth or width greater than 12 inches, the maximum stress for the member subject to bending is to be reduced. However, the strength is increased based on the unbraced length of the member. Since this study is intended to only study the feasibility of the Beverly Bridge and the exact bracing, temperature, and moisture conditions of the bridge are unknown, no strength increases or decreases were applied to the sections. The cross-sectional areas and section moduli of the elements were reduced where timbers were notched at the connections

Loading for the bridge was determined based on the 2012 AASHTO Bridge Design Specifications (AASHTO 2012). The HS-15 load represents a 15-ton truck, which is standard for bridges that are not on major traffic routes or federal highways. Since only a two-dimensional model of the bridge truss is analyzed, it is assumed that the 15-ton load is distributed evenly between the two trusses, resulting in a 15-kip point load directly at a truss node (one ton is equal to two kips). The bridge is tested for this live load at the midpoint of the truss, quarter point, and node nearest the support. A second case specified by the AASHTO code is a distributed lane loading combined with a moving point load. The value of the lane loading is 480 pounds per foot, and the point load is 13.5 kips total for moment (which was used at the midspan and quarterspan) and 19.5 kips for shear, which was applied at the node nearest to the support. The three load cases are shown in Figure 3.1 below.

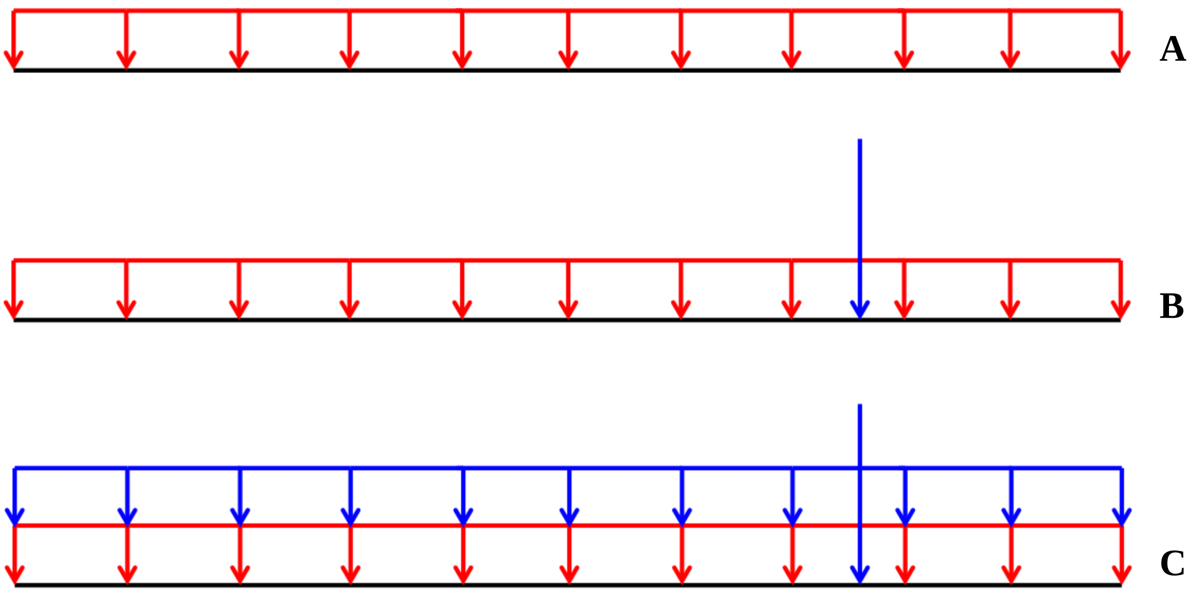

Figure 3.1: Bridge Loading: A: self weight only, B: self weight plus 30 kip moving point load, C: self weight, lane load of 0.48 kips per foot, plus a concentrated load of 13.5 kips for moment, 19.5 kips for shear

Dead loads were based on the self weight of the truss members (applied uniformly in the finite element analysis and at the nodes in the simplified analysis), and assumed weights of siding, roofing materials, floor framing, and lateral bracing applied at the nodes (see Appendix E for unit weights used). Material unit weights were based on the NDS standards and it was assumed that yellow poplar is used for all members, excluding the diagonal truss members. The AASHTO code specified a dead load of 50 pcf for timber, which is 66% higher than the tested density of yellow poplar. The AASHTO specified value is based on the assumption that pressure-treated or chemically preserved lumber is used, which can have a higher unit weight than standard sawn lumber (Ritter 1990). For the trusses, the self weight is distributed along the length of the member for the finite element study and split between each of its end nodes. Weights were calculated by multiplying the cross-sectional area of members with the material self weight. Deck cross sections were estimated using the values of deck thicknesses given by Kemp and Lamar.

The connections between timber elements required that several assumptions about the connectivity be made in analysis. It is difficult to connect timber elements such that they transfer axial force only, as almost all timber connections transfer axial load, shear, and flexure (Pierce 1999). All connections were assumed to be fully moment released in the simplified study, which reduces the truss to a statically determinate structure. Since the truss chords and arch consist of two parallel members with staggered splices, they were analyzed as continuous members in the finite element study. For the finite element analysis, three different connectivities were studied. In the first, all connections were modeled as fixed. In the second study, moments were released at every connection of the diagonals and verticals. The final study is probably closest to the actual behavior of the truss. Moments were released only at the ends of the diagonals and verticals, but those members were continuous elsewhere. This would partially account for the fact that these members are continuous through the arch in actuality. In the simplified analysis, only axial forces were transferred between the members. The arch and truss are modeled in two dimensions in all studies and out of plane effects are ignored. This assumption is valid because the top chord would be braced by the lateral load resisting system and the bottom chord would be restrained by the platform. The highest stressed unbraced diagonal member is checked for buckling.

Three simplified analyses were performed with dead load and a 15 kip point load at the midpoint, quarter point, and node nearest a support. All of these analyses assumed that the truss and arch spans were identical, meaning the bay bearing on the abutment plays no role in the structure. However, this bay was included in the finite element analysis and the truss was modeled as simply supported at its ends. The first analysis studied the stresses in the truss members if the arch carries no load. The second analysis studied the axial forces if the arch only carries its self weight and the truss dead load. The truss is assumed to only support the 15 kip point loads. In the final study, Lamar and Schafer's conclusion that the arch carried three times as much load as the truss is tested (Lamar and Schafer 2004). Three quarters of the uniform dead and live load is applied to the arch, and the truss carries the final 25% of the uniform load and all of the concentrated live load. The total uniform load capacity of the bridge is then calculated using various ratios of load carried by the truss versus the arch.

The finite element study was computed using a two-dimensional model of the truss in the software SAP2000 (CSI 2011). Axial forces, moments, and shear forces were calculated under each load and under each connection scheme. The combined stresses were then compared to the material capacities. In addition, total deflection of the bridge was studied for each case and compared to the anticipated long-term deflection.

3.3 Simplified Gravity Analysis

The first case studied in the simplified analysis assumed that the arch carries no load, meaning all dead and live load is applied to the truss at each node along the bottom chord. Key assumptions for the analysis were outlined in Chapter 3.2. Results are given in terms of the member's axial force divided by its capacity, meaning a value above 1 corresponds to an overstressed member. The location given is the panel number starting with Bay #1 at the support, and Bay #8 at midspan. Results are summarized in Table 3.3.

Table 3.3: Truss Only Simplified Analysis Results. Values are in terms of combined bending and axial demand divided by the capacity.

Load Case	Maximum Tension	Maximum Compression	Diagonal Compression
Dead Only (Location)	1.13 (Bottom, #8)	1.16 (Top, #8)	0.68 (#2)
15-Ton Midpoint	1.61 (B#8)	1.62 (T#8)	0.84 (#2)
15-Ton Quarterpoint	1.37 (B#8)	1.45 (T#8)	0.83 (#2)
Lane+6.75-Ton Midpoint Load	1.98 (B#8)	2.01 (T#8)	0.98 (#2)
Lane+6.75-Ton Quarterpoint Load	1.87 (B#8)	1.94 (T#8)	1.02 (#2)

It is clear from this analysis that the bridge would be well overstressed without the addition of the arch. The top and bottom chords are both overstressed under the bridge's own self weight, as the unity check is greater than one for all load cases studied. The truss performs better under the HS-15 concentrated load than under the distributed lane load combined with the point load. Under the lane loading case, both the top and bottom chords are loaded with nearly double its allowable stress. This is also the only case where the truss diagonal element is overstressed. The following cases study different simplified assumptions of the combined arch-truss behavior.

The first combined case studied the assumption that the arch carries the entire uniformly distributed load. Arches are generally more efficient than trusses under distributed loading, while trusses perform much better than arches under concentrated loading. The loading is applied to the truss at each point where it crosses the vertical truss elements. It is also assumed that there is no bending in the arch, since it is braced by the truss. Results for this analysis are summarized in Table 3.4.

Table 3.4: Arch Carries Uniform Load, Truss Carries Concentrated Load (Demand divided by Capacity)

Load Case	Maximum Tension	Maximum Compression	Diagonal Compression	Arch Compression (Thrust)
Dead Only (Location)	0	0	0	0.60 (84.5 kips)
15-Ton Midpoint	.49 (Bottom#8)	0.44 (Top#8)	0.17 (#2)	0.60 (84.5)
15-Ton Quarterpoint	0.34 (B#5)	0.37 (T#6)	0.25 (#2)	0.60 (84.5)
Lane+6.75-Ton Midpoint Load	0.36 (B#8)	0.36 (T#8)	0.17 (#2)	0.95 (133)
Lane+6.75-Ton Quarterpoint Load	0.28 (B#6)	0.3 (T#6)	0.19 (#2)	0.95 (133)

Under this analysis case, all truss elements are well within their capacity, as the highest stress is reached under the 15-ton midpoint load, where the bottom chord is stressed to almost half of its capacity. Under the self weight, all elements are within their capacity. However, under the uniform lane load plus concentrated load, the arch is stressed nearly to its capacity, with a stress ratio of 95%. Another notable result is that the maximum stressed members are located at the concentrated loads in this case. The maximum loads across all cases occur when the concentrated load is at the midspan. However, this load case is unlikely to occur in actuality, as the truss would take some of the load away from the arch. This situation is realized in the third analysis case.

The study of the Pine Grove Bridge (Lamar and Schafer 2004) suggested that the arch carries up to three times the load of the truss. This was represented by applying 75% of the dead load and uniform lane load to the arch, and the remaining uniform load and the entire concentrated load to the truss. Results for this analysis case are summarized in Table 3.5, below.

Table 3.5: Arch Carries 75% Uniform Load, Truss Carries Remaining Loading (Demand divided by Capacity)

Load Case	Maximum Tension	Maximum Compression	Diagonal Compression	Arch Compression (Thrust)
Dead Only (Location)	.28 (Bottom #8)	0.29 (Top #8)	0.17 (#2)	0.46 (61.6 kips)
15-Ton Midpoint	.77 (B#8)	0.74 (T#8)	0.33 (#2)	0.46 (61.6)
15-Ton Quarterpoint	0.57(B#5)	0.60 (T#6)	0.42 (#2)	0.46 (69.4)
Lane+6.75-Ton Midpoint Load	0.66 (B#8)	0.65 (T#8)	0.35 (#2)	0.72 (95.9)
Lane+6.75-Ton Quarterpoint Load	0.55 (B#8)	0.59 (T#8)	0.38 (#2)	0.72 (95.9)

In this case, when a portion of the uniform load is distributed along the arch, member stresses fall below their capacities under all loadings. The stress in the arch is now 28% below its capacity, whereas it was nearly overstressed when it supported the entire uniform load. Additionally, the truss is closer to its maximum capacity. Under this scenario, truss stresses are highest under the 15-ton concentrated load, and the arch is most stressed under the uniform lane loading. The highest thrust from the arch under this case is 96 kips, whereas a thrust of 133 kips would occur if the arch carried the full uniform load.

A final study was performed in order to determine the maximum total load that can be carried by the system. According to Ron Anthony, a wood scientist at Anthony and Associates, ultimate strengths of timber structures could be evaluated by different methods than the design strength method shown above (personal communication with Anthony, 2014). The arch and chords are both composed of parallel timbers, which add redundancy and resilience to the system. It would be reasonable to assume that the overall ultimate strength of these elements is higher than the maximum stresses given in the National Design Specification (Ibid.). However, since no specific strength values for the timbers are available, the design strengths are used to develop a conservative estimate of the ultimate capacity. Various values of percentage of load carried by the truss were used. The results of this study are shown in Figure 3.2.

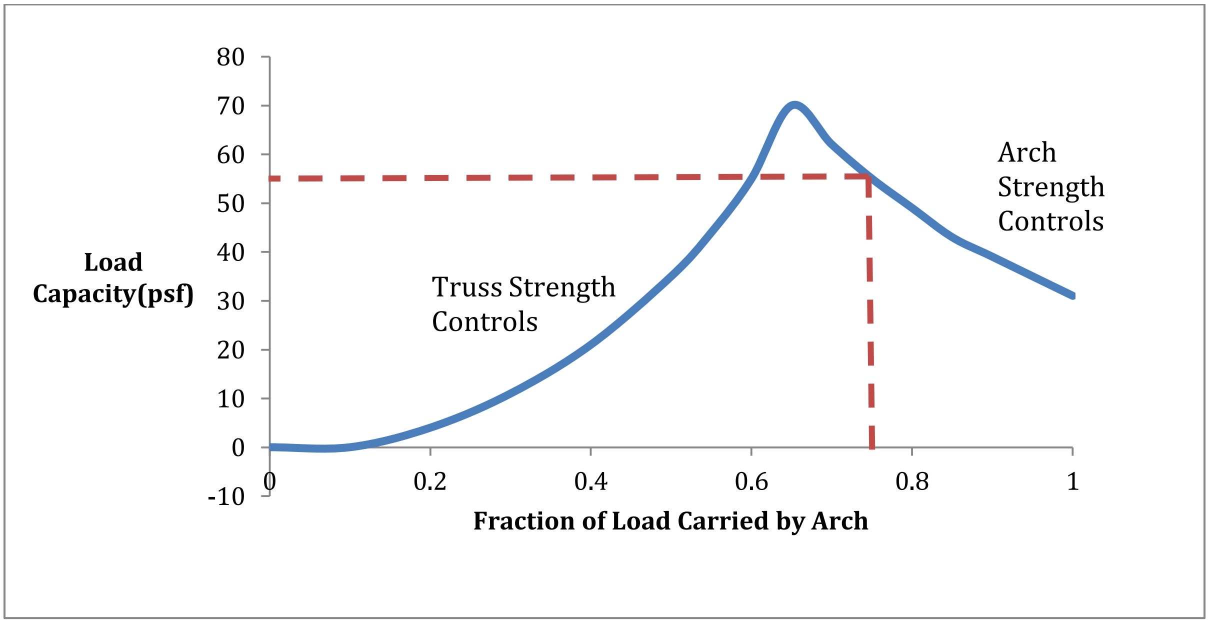

Figure 3.2: Beverly Bridge Load Capacity

At the assumed value of 75% of the weight distributed to the arch, the load capacity was found to be approximately 55 psf. At this point, compressive strength of the arch section controls the capacity. The above chart (Figure 3.2) shows that once the arch capacity is reached, additional load could be distributed to the truss. If this is the case, an ultimate load capacity of 70 psf could be attained. The critical portion of the truss is the bottom chord at the middle of the span, which fails in tension.

This ultimate load capacity does not account for the resiliency inherent in the system. Material strengths are based on a failure probability derived from physical testing. The design strength of the material is based on a specific probability of failure. If two members are used in parallel, the system will have a smaller standard deviation of strength and the same mean strength per unit of area. This means that the failure probability used to determine the design strength will occur at a higher strength value for two parallel members than for a single member with the same total cross-sectional dimensions. This is similar to the concept that a glulam member of a given species will have a higher design strength than a sawn timber member of the same size. In this ultimate load test, failure occurs at either the bottom chord or in the arch, both of which consist of two parallel members. It stands to reason that the actual ultimate load is significantly higher than the value calculated above.

The simplified model is also tested against a finite element model of the combined system, which is discussed in the following chapters. Overall, it is clear that load must be distributed between both the arch and the truss in order for the Beverly Bridge to carry traffic. Some sources, such as the Federal DOT Covered Bridge Manual, discuss analyzing the combined arch-truss with the assumption that the arch is not load-bearing and only serves to brace the truss and prevent long term deflection. It is clear that this method is overly conservative, as the load capacity of the arch is essential to the overall load capacity of the bridge.

This simplified analysis showed that the Beverly Covered Bridge would be stressed beyond its design limit and could fail under live loading if it were not reinforced with a timber arch. If the arch carried the uniformly distributed applied load, it is stressed to 95% of its capacity. When 25% of the uniform load is applied to the truss, both the arch and truss are within their design capacities under HS-15 loading. The ultimate capacity of the structure was also tested. If the arch carries 75% of the load, then an applied surface pressure of approximately 55 psf would be the absolute limit. This value is well above the typical mid-19th century bridge design loading of 30 to 35 psf (Kemp 1975). One key limitation of this study is that bending moments in the members are assumed to be negligible. The finite element analysis carried out in the following chapter attempts to address this shortcoming.

3.4 Finite Element Gravity Analysis

A linear static analysis of the Beverly Bridge was conducted using SAP 2000, using the geometry given in Chapter 2 and the material properties from the National Design Specification, which were detailed in Chapter 3.2. This study was conducted because the conclusions reached by Lamar and Schafer were that the combined arch-truss may be significantly stronger than the sum of the two separate systems. No finite element analysis would be able to establish the actual internal forces in the structure due to the static indeterminacy of the system and the sensitivity to small, unknowable changes in the boundary conditions. Different construction sequences can lead to different load paths. Additionally, since timber is a highly variable material, some members may be stiffer and stronger than others, causing loads to redistribute. According to Darryl Weiser, an experienced covered bridge builder would select the strongest timbers available for the arch, meaning the arch may take more load from the truss than the following analysis would conclude. Finally, long-term behavior of timber can greatly alter the stresses. A new timber bridge would likely be cambered by over 12 inches, and the initial stresses and the stresses after creep and wood shrinkage occur could vary widely.

Since the exact connection rotational stiffnesses are unknown, various connectivity schemes were tested in the SAP2000 analysis in order to establish a range for the possible internal stresses. For each case, the three loading types discussed in Chapter 3.2 were applied. The combined stresses from axial force and bending in each case were calculated, and the maximum value for each loading type is given below. The force and moment diagrams for the structure show both the maximum and minimum value across all load cases for each connectivity.

Model 1: Fully Rigid

The first connection scheme tested modeled every truss connection as fully rigid. This case gives an upper bound on the moment transfer between the members, but this behavior is nearly impossible to realize. Axial force and moment diagrams are shown below in Figure 3.3 and Figure 3.4. The figures are only meant to show the overall behavior of the structure. The red represents the minimum absolute value of the force or moment, while the blue represents the maximum absolute value. Generally, the top chord and diagonals are in compression, and the bottom chord and verticals are in tension. Where the arch intersects the bottom chord, the vertical tends to go into compression in this model.

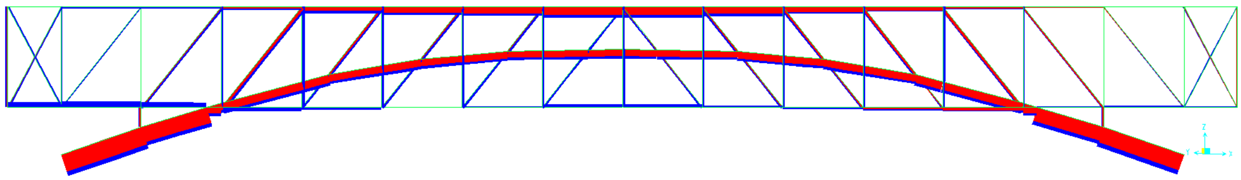

Figure 3.3: Rigid Model Axial Force Envelope

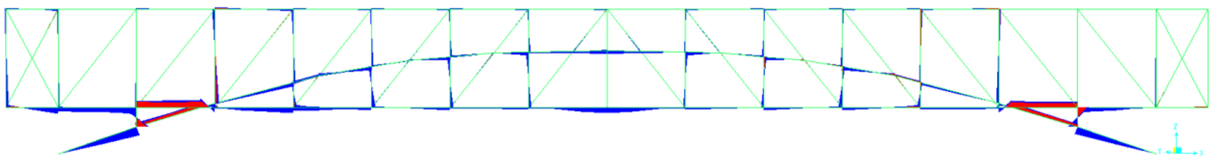

Figure 3.4: Rigid Model Moment Envelope

The highest axial forces are clearly in the top chord and in the arch near the supports. There are also moments present in the bottom chord, vertical members, and arch near the two support points. The arch sections where it falls below the bridge deck could have concerning bending moments because they are loaded with heavy concentrated forces and are no longer braced by the truss. For most of the structure, the truss braces the arch, so the bending moments are minimal. In this case, the maximum deflection of the structure is 1.03 inches, at the midpoint of the truss. A summary of the stresses in the structure is given in Table 3.6.

Table 3.6: Fully Rigid Model Results Summary (Demand divided by Capacity)

Load Case	Maximum Tension	Maximum Compression	Diagonal Compression	Arch Compression (Thrust)
Dead Only (location)	.40 (Vertical #5)	1.21 (Vertical #4)	0.26 (#4)	0.82 (80 kips)
15-Ton Midpoint	0.64 (V#5)	1.49 (V#4)	0.33 (#4)	1.03 (103)
15-Ton Quarterpoint	0.68 (V#5)	1.73 (V#4)	0.42 (#2)	1.12 (108)
15-Ton Support	0.48 (V#2)	1.34 (V#4)	0.29 (#1)	0.95 (87)
Lane+6.75-Ton Midpoint Load	0.77 (V#5)	2.01 (V#4)	0.42 (#4)	1.38 (137)
Lane+6.75-Ton Quarterpoint Load	0.81 (V#5)	2.22 (V#4)	0.49 (#4)	1.49 (146)
Lane+9.75-Ton Support Load	0.75 (V#5)	1.91 (V#4)	0.37 (#2)	1.33 (126)

Across every load case, the vertical member where the arch intersects the bottom chord carries a high bending moment, which leads to the member being overstressed. For example, lane load plus midpoint load case, the critical member carries an axial load of 13.4 kips, but carries a significant moment of 343 kip-in, so the member fails primarily in bending. The stresses in the critical verticals are generally higher than the stresses in the chord members. This behavior is likely unrealistic. The verticals and chords are connected by a single through bolt, so little moment would be transferred between the two.

The model predicts that the arch becomes overstressed due to the contribution from bending. In the case of the lane load plus the quarter point load, the axial load alone stresses the arch to 88% of its capacity.

The bottom chords show low stresses because the arch prevents the members from lengthening in tension. Forces in the top chord are generally higher than that of the bottom chord, but both members are within their capacity.

Model 2: Partially Rigid

The second truss model studied is likely the closest to the actual behavior of the bridge. Moments are released at the ends of the diagonals and verticals. This model reflects the fact that these members are continuous through the arch and connections are pinned. Diagrams of the axial force and bending moment envelopes are shown in Figure 3.5 and Figure 3.6, respectively.

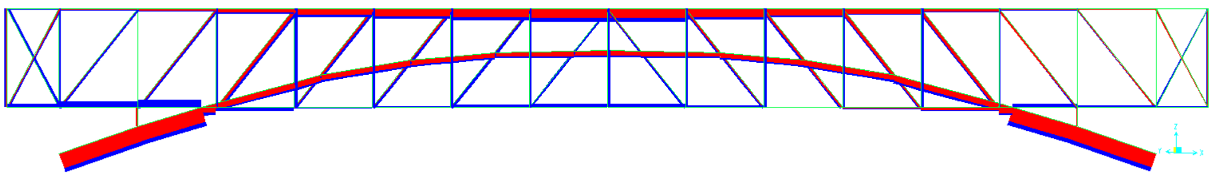

Figure 3.5: Partially Rigid Model, Axial Force Envelope

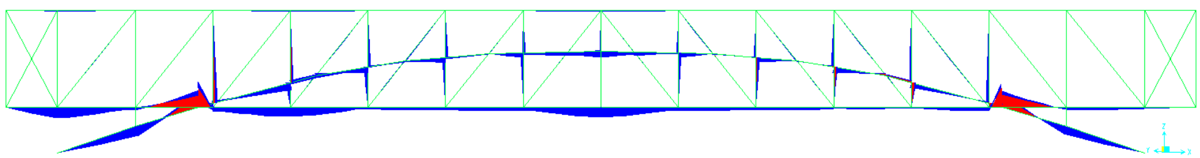

Figure 3.6: Partially Rigid Model, Bending Moment Envelope

In this model, deflections were similar to that of the fully rigid model. The maximum deflection across all loads was found to be 1.07 inches, nearly identical to the value of 1.03 inches above. Generally, the magnitude of the bending moments in the diagonals and verticals was reduced here. Under dead loading, the maximum moment was 145 kip-inches, located at the bottom chord, where it meets the arch. In the fully rigid model, the maximum calculated bending moment was 154 kip-inches, in a vertical member near the same location. Bending in the bottom chord is higher in this model, but the deep chords have a high bending capacity. A summary of the results is shown in Table 3.7.

Table 3.7: Partially Rigid Model Results Summary (Demand divided by Capacity)

Load Case	Maximum Tension	Maximum Compression	Diagonal Compression	Arch Compression (Thrust)
Dead Only (location)	.47 (Bottom #3)	0.53 (Top #8)	0.25 (#4)	0.76 (74 kips)
15-Ton Midpoint	0.64 (B#3)	0.81 (T#8)	0.32 (#4)	0.96 (102)
15-Ton Quarterpoint	0.81 (B#3)	0.63 (T#8)	0.41 (#2)	1.04 (107)
15-Ton Support	0.52 (B#3)	0.54 (T#8)	0.30 (#1)	0.85 (84)
Lane+6.75-Ton Midpoint Load	0.80 (B#3)	0.92 (T#8)	0.38 (#4)	1.22 (130)
Lane+6.75-Ton Quarterpoint Load	0.83 (B#3)	0.84 (T#8)	0.43(#4)	1.27 (125)
Lane+9.75-Ton Support Load	0.71 (B#3)	0.81 (T#8)	0.42(#2)	1.21 (125)

Under the self weight and 15-ton point loads, the only overstressed member is the arch under the quarterpoint load. The arch is overstressed under all of the lane load cases where the total load on the structure is much higher. It is important to note that the axial forces in the top chord of the truss are much higher than the forces in the bottom chord. The arches restrict the bottom chord from lengthening under tension, so this force is redirected into the top chord. Where the arch becomes overstressed, there is a combination of axial force and bending moment. This is clear from the force diagrams above, where the largest magnitudes of axial force and bending are present where the arch and bottom chord intersect.

The values of the thrust from the arch in the fully fixed model are slightly higher than in the partially rigid model, meaning that the arch carries a greater share of the load. However, the values are generally similar and the key difference between the two models is the elimination of bending stresses in the vertical members when the ends are pinned.

Model 3: All Connections Released

The final model tested consisted of continuous chord and arch members, but the verticals and diagonals are pinned at the connections to both the arch and the chords. The axial force and bending moment diagrams are shown in Figure 3.7 and Figure 3.8.

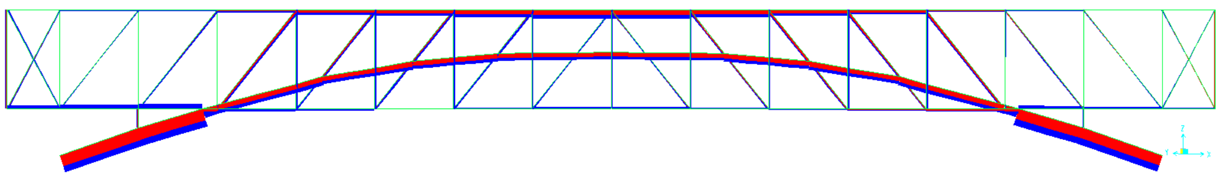

Figure 3.7: Fully Released Model Axial Force Envelope

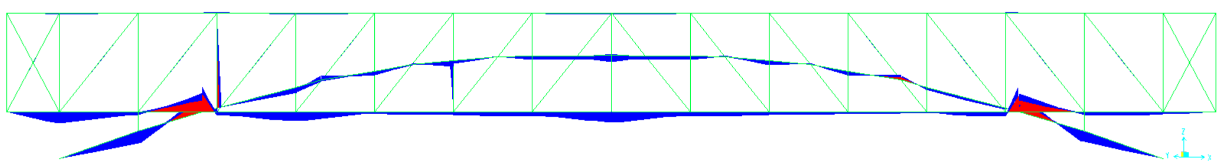

Figure 3.8: Fully Released Model Bending Moment Envelope

The results are generally similar to that of the partially rigid model, but it is possible that some members are higher stressed in this case. Also, the fully released model had the largest deflection of the three models, at 1.6 inches under the lane load plus midspan point load. Results of the analysis are shown in Table 3.8.

Table 3.8: Fully Released Model Results Summary (Demand divided by Capacity)

Load Case	Maximum Tension	Maximum Compression	Diagonal Compression	Arch Compression (Thrust)
Dead Only (location)	.39 (Bottom #3)	0.53 (Top #8)	0.25 (#4)	0.75 (78 kips)
15-Ton Midpoint	0.45 (V#4)	0.80 (T#8)	0.32 (#4)	0.95 (100)
15-Ton Quarterpoint	0.79 (B#3)	0.63 (T#8)	0.41 (#2)	1.03 (105)
15-Ton Support	0.49 (B#3)	0.54 (T#8)	0.30 (#1)	0.84 (85)
Lane+6.75-Ton Midpoint Load	0.61 (B#3)	0.92 (T#8)	0.39 (#4)	1.23 (113)
Lane+6.75-Ton Quarterpoint Load	0.77 (B#3)	0.84 (T#8)	0.43 (#4)	1.26 (133)
Lane+9.75-Ton Support Load	0.66 (B#3)	0.80 (T#8)	0.35 (#2)	1.19 (125)

Member stresses are generally similar to those in the partially rigid model. The arch stresses are lower, since no bending moment is transferred from the vertical members. Again, the arch becomes overstressed in the lane load cases.

Shear is not an issue in the structure, as the largest shear force is on the order of 10 kips and was in the bottom chord, which has a shear capacity of 21 kips.

Buckling was checked for the highest stressed diagonal member. The unbraced diagonal is classified as a "long column" based on its length to cross-sectional dimension. Buckling strength was determined using Equations 2, shown below.

$$F_c = \frac{0.3E}{({}^{l}/_{d})^2} \qquad \text{Eqn. 2}$$

In this case, F_c is the critical compressive stress, l is the unbraced length, and d is the length of the shortest cross-sectional face. In the case of the oak diagonals, this critical stress is

573 psi. The maximum stress in the unbraced member was 422 psi, in the case of the quarterpoint load. The allowable stress is clearly controlled by buckling; though the diagonals are not overstressed when this is taken into consideration.

The three models showed that bending stresses within the members are an important consideration in the Beverly Bridge's load capacity. The arch is loaded at discrete points along its length. These point loads lead to bending, which stress the arch beyond its capacity in the lane load cases. The arch is also stressed beyond its capacity under a point load at its quarterspan. The truss members were shown to be able to withstand modern vehicle loading as long as excessive moments are not transferred to the vertical members. It is possible that as the arch becomes heavily stressed, load can be transferred to the truss, which would have excess load capacity. The following sections will discuss lateral and dynamic loading of the Beverly Bridge, and the results from all analyses will be discussed further at the end of this chapter.

3.5 Lateral Analysis

A simplified lateral analysis was performed in order to obtain an estimate for the deflections and reactions at the abutments under wind loading. The 2012 AASHTO code was used to determine the value of the wind pressure. For a large flat surface, a value of 40 pounds per square foot is recommended. The surface area used is the height of the truss and the width of the face is the distance between the abutments. This uniform load is transferred to the lateral bracing at each truss node. For the first study, the bridge as it was likely originally built was considered. The only lateral load resisting system is the timber cross bracing. The second study considers a modification to the bridge of added steel cables beneath the bridge deck, as is shown in the HAER drawings of the Barrackville Bridge (HAER 2003).

The geometry of the two bracing schemes used in the analysis is shown below. Figure 3.9 shows the geometry of the bracing at the level of the top chord of the truss. The chords of this truss are the top chord of the gravity bearing truss, which consist of two parallel members measuring 10" x 6". No measurements of the bracing elements are available, so they are all assumed to be composed of yellow poplar members measuring 8" x 8" in cross section. Since this is an indeterminate system, a simplifying assumption that the shear in the truss is shared equally is made.

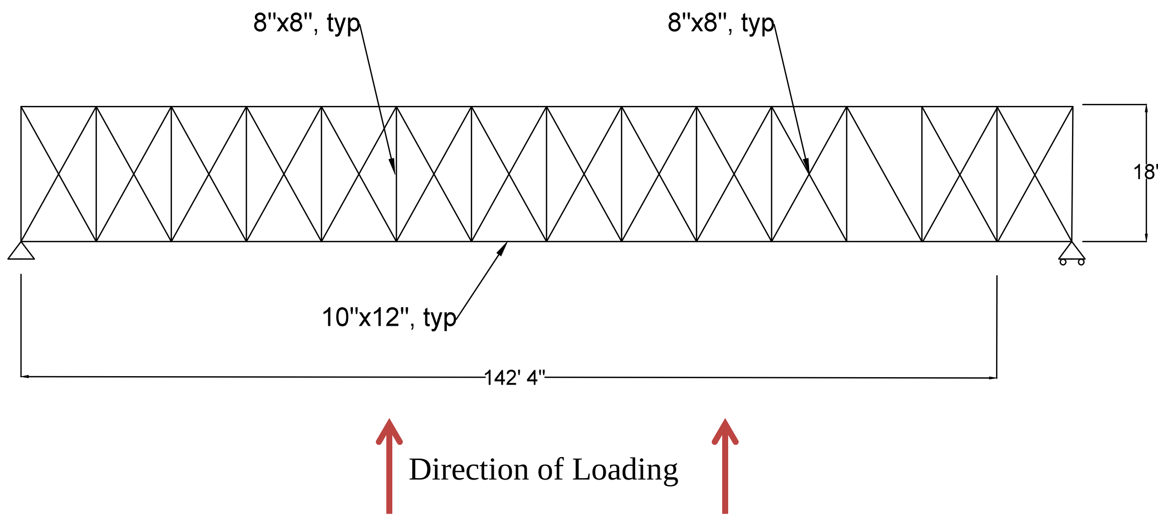

Figure 3.9: Top Lateral Bracing Scheme

At the Barrackville Bridge, cross bracing ties were added to stiffen the bridge against lateral loads. For the second analysis, it is assumed that the bracing trusses at the top and bottom of the bridge share the wind load equally. The geometry of the bottom bracing used in the analysis is shown in Figure 3.10. The chords of the truss are the bottom chords of the gravity truss, which are two parallel 7” x 14” poplar timbers. The vertical members of the truss are the floor beams, which were not measured and are assumed to be 24 inches deep and eight inches wide, though this size would be determined by a detailed gravity analysis of the bridge platform. The diagonal elements are one-inch diameter steel cables. Only one element is shown at each truss panel because the ties can only work in tension, so the bracing effectively acts as a Pratt truss.

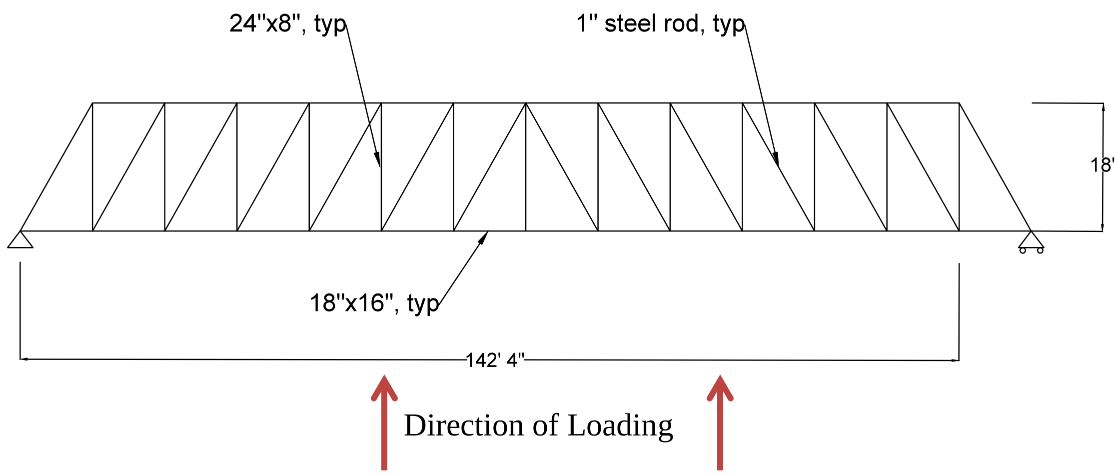

Figure 3.10: Bottom Lateral Bracing Scheme

Internal forces in the members were determined through a simple equilibrium analysis with point loads equal to 40 psf multiplied by the tributary surface area applied to each node of the trusses. Deflections were calculated using the principle of virtual work at the midpoint of the structure. Under this analysis, the reaction force at the abutments is 16.7 kips. Maximum stresses and deflections for the two cases are shown below in Table 3.9.

Table 3.9 Lateral Analysis Results

Case	Top Bracing Only	Top Bracing, Split Load	Bottom Bracing, Split Load
Top Chord Stress (ksi)	-0.66	-0.33	-0.12
Bottom Chord Stress (ksi)	0.66	0.33	0.12
Vertical Member Stress (ksi)	-0.26	-0.13	-0.08
Diagonal Stress (ksi)	0.30	0.15	24.5
Deflection (in)	0.70	0.35	1.16

Under the full wind load, the top bracing would be overstressed if it were the only component, considering that the maximum tensile stress for poplar is 0.575 ksi. The addition of bracing beneath the deck greatly reduces these stresses. Deflections are well within limits, as the maximum deflection at the bottom bracing is approximately 1/1500 of the span. Additionally, if the diameter of the steel rods were increase from 1” to 1.5”, this deflection is reduced to 0.66 inches. Further analysis would be required of the combined effects of gravity and lateral load on the structure, since it may be necessary to reinforce the chords of the truss if they are required to resist both loads.

3.6 Modal Analysis

A modal analysis was run on the finite element model to evaluate the bridge's performance under dynamic loadings. The partially rigid model was used to generate these results. In the case of the Beverly Bridge, the goal is to avoid resonance due to footfall excitation. For this reason, only response in the vertical direction on the two dimensional model was studied. Mass was assigned to the model based on the applied dead loads. A summary of the results is shown in Table 3.10.

Table 3.10: Mode Periods and Participation Factors

Mode Number	Frequency (Hz)	Participating Mass Ratio
1	0.14	0.72
2	0.48	0.11
3	1.04	0.01
4	1.19	0.07

A general range for walking frequencies is 1 to 4 Hz. The first two fundamental frequencies fall below this threshold, and these two frequencies account for 83% of the mass participation of the structure. The frequencies of modes 3 and 4 fall within the range of walking excitation, but their participating mass ratios are below 10%. Further study would be needed to analyze the bridge in three dimensions under both pedestrian footfall and seismic excitation. If the bridge were to be constructed for pedestrian use only, it is possible that vibration requirements would control the design of the timber deck.

3.7 Discussion and Conclusions

One goal of this study was to test simplified models of the truss behavior. Breaking the combined system into a statically determinate truss and a three-hinged arch may be useful in order to develop an estimate of a Burr truss's load capacity. The results from the partially rigid finite element model and the simplified model with 75% of the load carried by the arch are compared in Table 3.11.

Table 3.11: Results Comparison

Load Case	Simplified Model Unity Check	Simplified Model Case	FEM Unity Check	FEM Case
Maximum Compression	74%	Midpoint Load	92%	Lane Load plus Midpoint Load
Maximum Tension	77%	Midpoint Load	83%	Lane Load plus Quarterpoint Load
Arch Thrust: Dead Load	62 Kips	Lane Load	74 Kips	Dead Only
Arch Thrust: Dead+Live	96 Kips	Lane Load	130 Kips	Lane Load plus Quarterpoint Load

There are a few important shortcomings in the simplified analysis. Though the finite element model is by no means exact, it yields more accurate results than the simplified model. A primary reason is that bending moments in the arch were ignored in the simplified case. Since the arch is being loaded with point loads, bending is a major contributor to its stresses. This is a controlling factor near the abutments, where the truss does not brace the arch.

Modeling the truss as a standalone simply supported structure also leads to results that diverge from the finite element model's output. As one would expect, the largest stresses in the simplified model were at the chords near the midspan of the structure. These forces are typically of the same magnitude. The computer models show that the largest tension in the bottom chord occurs where the arch and chord intersect. The axial force in the top chord is largest near the center, as expected, but its value is larger than predicted by the simplified analysis.

The load capacity of the structure was studied based on various ratios of arch to truss tributary load. The finite element model shows that under uniform loading, the arch carries over three times the load carried by the truss. However, this ratio breaks down under live loading. Applied point loads to the truss are applied to the arch, but the bracing of the truss prevents the arch from deforming severely. It is clear that any simplified combined arch-truss model must dedicate a majority of the load to the arch, as it is the stiffer component, but an exact fraction would be impossible to attain. If three quarters of the load is carried by the arch, the structure would be within design stresses under a 55 psf uniform load, but this could go as high as 70 psf. However, a model that acknowledges the combined behavior would be necessary for evaluating the structure's behavior.

The structure of the Beverly Bridge generally does not have excessive deflections, as a live load deflection between one and two inches is likely. Initial camber could be on the order of ten times as large as this deflection. The structure can clearly support its own weight, and the only overstressed members under 15-ton point loadings were diagonals and verticals in the fully rigid model and the arch under quarterpoint loading. The arch also exhibits slightly higher than allowable stresses under these loadings, but demand is within five percent of the capacity. This model is unlikely to reflect the actual behavior of the structure, as full moments could not be transferred to these members. Only the arch was overstressed in the partially rigid or fully released models.

The arch of the truss were found to be loaded beyond its design capacities under the lane loading cases, which reflect a row of trucks bumper-to-bumper along the length of the bridge. These load cases correspond to a total load of 85 kips along the bridge. There are a number of uncertainties, so this result should not be taken as a sign that the Beverly Bridge would fail to meet AASHTO load requirements. For one, the bridge was modeled as simply supported from the end of the cross-braced bays. In actuality, these bays are bearing on the abutments, so the effective span of the bridge is shorter than it was modeled. Secondly, there are uncertainties in the dead load. The floor system of the bridge was not considered and could be lighter than its assumed load for the purposes of this analysis. Finally, the timber species used for the top chord and arch could be oak, which has a design compressive strength that is 22% larger than that of yellow poplar.

Further study of the bridge would be required to determine that exact load capacity of the Beverly Bridge. The connection strength was not tested, and the exact stiffnesses of Chenoweth's connections are unknown. Also, the location of splices could affect the behavior of the chords and arch, though these were modeled as continuous elements. The effect of various load combinations should also be evaluated. Wind or snow combined with live loading could control the design of the structure. Construction sequencing and member prestress should also be considered. The manner in which the bridge is constructed would affect the load path. Also, it is clear that Chenoweth constructed the bridge's overhead bracing in a manner that allows for prestress and adjustment. These adjustments could affect the stresses in the top chord, as some pretension could bring the chord stress below its critical value under the controlling live load cases.

A major limitation of the finite element analysis used was that the arch was modeled as a series of continuous beam elements. This does not account for the behavior of the arch at the splices, where some rotation between connected elements is possible and expected. This behavior would reduce the overall stiffness of the arch compared to how it was modeled. This reduction in stiffness would mean the truss, which generally has excess capacity, would carry that additional load. Rotation of connecting arch members would also reduce its internal bending moment, which may bring the demand below its allowable capacity.

If further study shows that the Beverly Bridge as originally constructed is not adequate for modern traffic demands, several modifications could be considered. The first would be a load limit. The historical photos show that the bridge had a weight limit of three tons. The analysis suggested that the bridge could resist a truck that is significantly heavier than this. Additionally, there is precedent for limiting the applied loads on historical bridges. For example, timber covered bridges in Pennsylvania have a load capacity of 6 tons (Lamar 2002). Limiting the bridge to carrying one truck at a time could allow the bridge to serve traffic on the same road where it was originally built. Also, modifications to the structure itself could be considered. The cross sections used for the top chord and arch were based on the dimensions at the Barrackville Bridge, but it is possible that members larger than this were used, especially considering that Beverly spans ten feet longer than Barrackville. Glue laminated members could also be used in place of sawn lumber as a replacement. This would increase the capacity of each member, without greatly affecting the bridge's appearance. Both structural and cultural concerns must be taken into account if the Beverly Bridge is to be rebuilt to modern bridge standards.

4 Reconstruction

The analysis detailed in Chapter 3 established that it is feasible for the Beverly Bridge to carry modern traffic and that further study would be required to see if any minor modifications would be required. Further study also would be required of the connection properties to properly gauge the internal stresses. Additionally, the analysis did not include study of the bridge platform framing, which must be able to withstand wheel loads. In the following section, an overview of bridges similar to Beverly is given, which shows that it is feasible for similar structures to serve traffic demands today. As an example, the Smith Millennium Bridge is discussed in detail, since its reconstruction can serve as a model for Beverly. Potential modifications to the Beverly Bridge, both structural and non-structural are considered. Finally, the cost of rebuilding the bridge today is estimated.

4.1 American Burr Truss Bridges

It is estimated that by 1885, ten thousand covered bridges were in use across the United States (Pierce 1999). The National Society for the Preservation of Covered Bridges has published a thorough catalogue called the *World Guide to Covered Bridges*, most recently in 2009. This publication details the location, span, date of construction, truss scheme, and status of every covered bridge worldwide. As of the most recent publication, there were 814 covered bridges standing in the United States. Of this pool, there were 44 combined arch and truss bridges that had a span of over 130 feet. Twenty one of these currently carry traffic, and eight were either newly constructed or completely rebuilt after 1970. Of the bridges in that subset that currently carry traffic, the oldest was built in 1852 and the longest span measures 198 feet. It is unclear whether this includes truss elements that extend past the abutment face. The complete listing of these bridges is shown in Appendix A. It is clear that there is a precedent for 150-foot span Burr Trusses such as Beverly carrying traffic in rural areas over the course of decades. The Beverly Bridge could show the same longevity, as it carried traffic demands up until it was destroyed in 1951.

The Smith Covered Bridge serves as a strong point of comparison for how the Beverly Bridge could be rebuilt. The original Smith Bridge was built over the Baker River in Plymouth, New Hampshire in 1850 (Pierce et al. 2005). It was destroyed by arson in 1993, and a new covered bridge (known as the Smith Millennium Bridge) was constructed from 2000 to 2001.

Like the original, the new bridge was constructed entirely from timber, and has an overall length of 176 feet, with a clear span of 150 feet (five feet longer than the Beverly Bridge). It has a Long Truss configuration and is supplemented with an arch.

Multiple modifications were made, so the Smith Millennium Bride is not an exact replica of the original. The historic bridge had a single, 18-feet wide lane, but the new bridge was built with two full lanes, each measuring twelve feet wide. The height of the trusses was raised to 14.5 feet in order to better accommodate larger trucks. An outboard sidewalk was added to the bridge, also. The bridge does not sit on the original abutments. The new abutments are reinforced concrete with pile supports. A granite facing was added to the abutments in order to match the original appearance.

The modern Smith Millennium Bridge was designed for the AASHTO HS-20 loading. It is the first covered bridge that meets this rating. This design loading required the use of southern pine glulam timbers, which have a much higher reliable strength than the equivalent sawn timbers. The deck is composed of 6.75-inch thick glulam, and a replaceable oak wearing surface was added on top. Thirty inch deep glulam beams support the deck. The bridge also has a fire alarm system, and most of the timbers are coated with a fire retardant.

The full bridge was constructed on falsework that had been erected in the river. Most of the bridge was assembled by lifting one member at a time into place with a crane. The total construction cost was $3.1 million, which includes the construction of the abutments and the realignment of a portion of the connecting highway. The superstructure alone cost $1.5 million. Of this, $1.4 million was used for the procurement and fabrication of the glulam members. The construction was performed by 3G Construction Inc, the firm of the Graton family of bridge builders. An image of the completed bridge is shown below in Figure 4.1.

Figure 4.1: Smith Millennium Bridge (gratonconst.com)

4.2 Structural Modifications

The first structural modification to consider would be the use of glulam in place of sawn lumber. One reason for this change is the limited availability of suitable sawn timbers. Old growth poplar trees are only standing in protected areas (Personal communication with Weiser, 2014) and the dimensions of lumber required would be difficult to find. Glulam could be manufactured with smaller source timbers. As a point of comparison, the axial compressive strength of Southern Pine glue laminated timbers can range from 1.5 to 2.4 ksi, whereas the strength of sawn Southern Pine lumber varies from 0.525 to 1.3 ksi, depending on the grade (NDS 2005).

Another important consideration is the long-term deflection of the bridge. If sawn lumber were used, construction would be done with green timbers. Shrinkage occurs as the members dry out, which could cause movement of the bridge. Additionally, wood tends to creep over time. According to Graton, a bridge with the span of Beverly would be built with an additional camber of at least 14 inches.

Modifications to the overall truss geometry would also need to be considered. The bridge was analyzed for this study with a depth of 12 feet, 8 inches. The Smith Millennium Bridge was constructed with a depth of 14 feet, 6 inches. A similar height may be required for the Beverly Bridge, in order to allow for adequate vehicle clearance, an important consideration given that the original bridge was torn down due in part due to insufficient clearance.

The width of the bridge may also need to be modified. The original bridge had a single 18 foot wide lane. This width would not allow for two simultaneous lanes of traffic. The bridge that replaced Chenoweth's work has a lane running in each direction. It is possible that local officials would refuse to build a bridge in Beverly's location that reduces the road's functionality. Either traffic would need to be restricted to one direction at a time, or a bridge wider than the original Beverly Bridge would need to be constructed in order to retain the current functionality. In the case of the Smith Millennium Bridge, the new structure was built with 24 feet between the trusses, allowing for two lanes of traffic.

The use of the abutments must be considered. The original stone abutments are still in use and have been partially covered with concrete. This concrete could either be stripped from the abutments or be incorporated into the structure. Using the reinforced abutments would shorten the span, which may improve the performance of the bridge, but this would come at the cost of historical authenticity.

Finally, pedestrian access should be considered. Many covered bridges have a pedestrian walkway on the side of the structure. If there is only a single traffic lane, a portion could be devoted to traffic, leaving the remainder for pedestrians. Otherwise, a separate platform would need to be added to ensure pedestrian safety.

4.3 Nonstructural Considerations

Aspects beyond the structure of the Beverly Bridge would need to be considered in order to allow for its construction. One consideration would be the approaches to the bridge. If it were only a single lane, additional safety measures would have to be implemented. Stoplights could be added at the approaches on each side of the bridge to ensure that two vehicles do not try to cross in opposite directions simultaneously. The road that the bridge was a part of is not heavily travelled, so traffic would not be greatly impacted.

Lighting would also need to be installed beneath the cover. This is standard practice in modern covered bridges. The HAER drawings of the Beverly Bridge from 1973 show electrical conduits and control boxes in the elevation of the bridge.

Fire protection is another concern specific to timber bridges. Sprinklers were installed in the Philippi Bridge after an accidental fire caused damage to the cover. In the case of the Smith Millennium Bridge, the structural timbers were treated with a fire retardant. According to Graton, many covered bridges today have fire protected timbers, sprinkler systems, or fire alarms. “Dry sprinklers” could be used if necessary, which avoids the concern that sprinkler pipes may freeze in the unheated bridges during the winter months. Some bridges are also outfitted with security cameras in order to avoid vandalism.

A final nonstructural concern integral for the preservation of a timber covered bridge is the prevention of water damage. The areas near the entrances of the bridge would need to be carefully maintained in order to control runoff (Pierce 1999). Stormwater must be contained and curtailed away from the bridge in order to prevent runoff from flowing onto the timber surface and coming into contact with the structural timbers. This introduction of moisture to the structure would over time lead to rot and premature strength reduction of the structure. The cover of the bridge must also be maintained in order to serve the same end. The siding should be replaced roughly every 30 years, or when excessive deterioration causes the cover to no longer serve its protective purpose. In the case of the Smith Millennium Bridge, a corrugated metal roof was used in place of a traditional shingle surface. Though a solution such as this is not authentic to the original bridge, it may be a cost effective means of extending the structure’s useful service life and reducing maintenance costs.

4.4: Cost Estimation

A major obstacle to rebuilding the Beverly Bridge is funding for construction. The following section attempts to quantify the costs for rebuilding the bridge, and compares this cost to that of a typical modern day bridge construction. According to JR Graton, one of the builders of the Smith Millennium Bridge, a single-lane arch-truss bridge such as Beverly would cost $1.5 to $2 million if funded by a state agency. This price depends on a number of factors such as the load rating and magnitude of abutment work. If private funding is used, this cost could go as low as $750,000, due in part to reduced letting cost.

The cost figures from the construction of the Smith Millennium Bridge were used in obtaining a second cost estimate. The Smith Bridge has a considerably larger roadway surface area than the Beverly Bridge, but it was assumed that the cost of superstructure materials is the same per square foot of roadway area. This is despite the fact that the Smith Bridge was built to withstand an HS-20 loading, while the preceding sections assumed the Beverly Bridge would be built to meet the HS-15 loading. Information about the Smith Bridge cost and dimensions are from Pierce, 2005. The CPI index provided by the Federal Reserve was used to compute the inflation over time. A complete breakdown of the cost estimation is shown in Table 4.1, below.

Table 4.1: Smith and Beverly Bridge Construction Comparison

Item	Smith Millennium Bridge	Beverly Bridge
Labor Cost	$100,000	$100,000
Material Cost per Square Foot, 2014 dollars (Minneapolis Federal Reserve)	$442	$442
Material Cost per Square Foot, Actual	$332	$442
Roadway Area (Square Feet)	4224	2970
Total Superstructure Cost	$1,500,000	$1,334,000

The estimated cost of the Beverly Bridge superstructure is lower than the cost estimated by JR Graton, but does not account for substructure work. To get a point of reference for these costs, the construction cost of a similar steel or concrete bridge must be estimated. The New York State Department of Transportation has the low bid costs of all new bridge and bridge replacement projects in the state from 2005 to 2012 publicly available. Bridges that would be similar to a replacement for Beverly were extracted from this set. The considered bridges spans range between 127 and 158 feet and are between 29 and 55 feet wide. A complete listing of the considered bridges is shown in Table 4.2.

Table 4.2: New York State Bridge Construction or Replacement Bids

Year	Length (ft)	Width (ft)	Cost ($)
2005	139	36.7	818,000
2005	144	29.3	1,197,315
2005	138	38.2	1,129,963
2005	157	53.1	3,966,983
2006	131	42.5	1,901,307
2006	127	42.6	914,513
2006	127	42.6	789,751
2006	131	42.5	841,729
2006	137	54.4	1,167,288
2008	127	36.0	1,255,886
2009	131	42.6	1,460,000
2010	134	44.9	1,725,000
2011	136	31.9	1,045,000
2011	157	52.3	2,043,000
2011	157	52.3	1,680,000
2012	136	31.9	987,000

The cost of the new bridges generally ranges between $790,000 and $2 million, with one outlier at $4 million for a bridge that was a part of Interstate 95. The average cost for the considered bridges is $1.43 million. It should also be noted that the CPI index increased by 14% over the time period considered. This cost is slightly higher than the estimated superstructure construction cost, but below the range given by JR Graton. This value also does not account for the cost of maintenance over time. Another factor is that the widths given are generally larger than that of the Beverly Bridge, which measures 18 feet. This discrepancy is due to the fact that there were not any single lane modern bridges listed, and many of the bridges not constructed incorporate a shoulder lane. It must be stated that this economic study cannot account for the cultural value that would be added to the community by rebuilding the Beverly Bridge.

5 Conclusions

The following chapter summarizes the results discussed above and suggests future work on Lemuel Chenoweth and the Beverly Bridge. The aim of this study was to establish the structural properties of the now demolished Beverly Bridge and evaluate its performance under modern loading. The cost of reconstruction of the bridge was also explored.

5.1 Chenoweth's Place in History

Timber covered bridges served an integral role in American commerce over the beginning half of the 19th century and are still a feature of rural areas across the country, especially in the Northeast and Mid-Atlantic regions. The development of the bridges, from Palmer's "Permanent Bridge" to all-iron Pratt trusses is well documented, but no comprehensive studies of the structures of Lemuel Chenoweth exist. His contributions, both to the history and culture of West Virginia and to technology, played a key role in the development of West Virginia.

Chenoweth clearly followed in the footsteps of Lewis Wernwag, the builder of the longest span timber bridge ever constructed in America. The exact manner of the relationship is unclear, and more historical research would be required in order to shed light on their past. Multiple sources suggest that Wernwag was the designer of the covered bridge at Beverly, located footsteps away from Lemuel Chenoweth's home. If this is not the case, then Wernwag was a key influence on Chenoweth. Prior to the construction of Beverly, Wernwag built a bridge over the Cheat River, 60 miles away from Beverly. Both bridges were situated on the same turnpike. Since Chenoweth had no experience in bridge construction prior to Beverly, it is possible that the Cheat Bridge was used as a point of reference. Details of Chenoweth's construction bear resemblance to what has been documented of Wernwag's bridges. One major similarity is the use of a single iron bolt at each connection, rather than the traditional all-timber joinery. Since no bridges built by Wernwag are currently standing, the bridges of Chenoweth can serve as testaments to their strength. Taken separately, the bridges of Chenoweth have shown remarkable longevity, as they were, for decades, able to carry loads far greater than they were initially designed for.

The bridge at Beverly was destroyed in 1951 and is an unfortunate lost part of Chenoweth's legacy. It was the first bridge built by Chenoweth, and was destroyed during the

Civil War. The bridge was rebuilt on the same abutments in 1873. This reconstruction was the final bridge project completed by Lemuel Chenoweth. By that time, he had built over a dozen bridges across the state, and others (such as the O'Brien brothers) had adapted his technology and construction methods. A third version of the bridge would honor the legacy of Chenoweth and the history of the region.

Unfortunately, no drawings or specifications of the bridge exist. Properties of the bridge were estimated using salvaged timbers, historical photographs, and surveys of Chenoweth's standing bridges. The first goal of this thesis was to establish the original dimensions and qualities of the bridge. Photographs of the bridge confirmed historical records which state that the bridge was a one lane, king post truss with an arch. Salvaged timbers were used to determine wood species and member dimensions, and the gaps in knowledge were filled in by studying Chenoweth's Barrackville Bridge. It is believed that the bridge studied is an accurate representation of the original bridge, with a margin of error on some of the member cross-sectional dimensions.

5.2 Structural Feasibility

Once dimensions and member cross sections of the Beverly Bridge were established based on similar precedents and historical records, an analysis of the arch-truss structure was performed in order to gauge its load capacity and performance under modern AASHTO loadings. The analysis is only intended to be a rough estimation in order to identify areas of concern and determine if it is feasible for the bridge to carry traffic with few modifications. For the timbers, strengths were conservatively determined using the National Design Specification critical values for sawn lumber.

The load transfer properties of the connections are unclear, so several models of the structure were tested. It was discovered that if significant bending moment is transferred between the chords and the vertical members, the verticals could fail in bending. Otherwise, the structure appears to be able to support its own self weight in addition to a 15-ton moving load on the structure. One concern is that a high point load could cause bending in the arch, stressing it beyond its capacity. The combination of the arch and multiple king post truss proves to be stronger than merely the sum of its constituent parts. The truss distributed the load along the length of the arch while bracing it against excessive deformations.

The analysis also showed that the bridge either exceeds or is close to exceeding its capacity under a full lane load. The arch is stressed almost 30 percent above its capacity. A combination of wind and vehicle loading could increase the stresses in the chords, possibly bringing them above their allowable loading. This shows that the proposed geometry and materials for the Beverly Bridge would not be able to carry full HS-15 loading. The splices in the arch may reduce bending stresses below the calculated values, so further study of load transfer through the arch would be necessary before construction. If a sawn lumber species other than poplar were used for the arch, load capacity could be increased without sacrificing historical authenticity. Also, it would be possible to modify the structure without being too intrusive while greatly improving its serviceability.

The structure of the bridge could be modified in order to improve its performance. First off, the sawn lumber used in this study could be replaced by glulam, which would increase the strength while maintaining the same dimensions and visual authenticity. A more historically accurate reconstruction would use sawn lumber, and a hardwood with a higher allowable stress than yellow poplar could be used to increase the capacity. Also, since the top chord is stressed nearly to its capacity, it could be constructed using oak instead of yellow poplar, which would increase its strength by 22%.

Under the same loading, the arch becomes overstressed due to a combination of bending and axial compression. Further study would be required in order to validate these results. The arch construction allows for some rotation at the splice locations, and this was not accounted for in the analysis. It is possible that some slight rotation relieves the bending stresses while maintaining axial rigidity. Also, the structure could be modified in order to distribute load more evenly to the arch. A common practice in similar structures involves adding iron components between the arch and the truss in retrofits. This is evident in the HAER drawings of the Barrackville Bridge. Additional members could also be added to the structure near the support, where the arch is below the bottom chord. This was the area that showed the most significant bending. A more uniform load application could reduce bending, and the additional members could be added below the cover, where they would not be externally visible.

If the bridge cannot be made to satisfy full AASHTO loadings without excessive modifications, the loading on the bridge could be limited. The analysis suggested that the bridge would be able to carry a 15-ton truck with some excess capacity, so the bridge could be limited to one vehicle at a time. It is located in a rural area and the road is not a major shipping route, so this would be unlikely to greatly impact traffic. The bridge could also have a posted vehicle weight limit, as it did in the past. The structure was able to withstand vehicles up to three tons in the past. Also, some states (such as Pennsylvania) limit the vehicle weight on covered bridges to six tons, which could in all likelihood be carried by the bridge at Beverly.

A secondary goal of the study was to compare a finite element analysis with a simplified analysis, performed by separating the arch-truss combination into two statically determinate structures. The DOT Covered Bridge manual suggests that the arch could be seen as only serving to brace the truss. A second suggested method is to assign the live load to the arch and the dead load to the truss. The analysis performed did not agree with either method, which follows the results obtained by Lamar and Schafer in their analysis of the Pine Grove Bridge. The truss would not be able to support its own weight according to the analysis, so ignoring the structural capacity of the arch would be overly conservative. In fact, the arch carries a majority of both the live and dead load of the structure. It is likely that the arch must be able to withstand 70% to 80% of the total load of the structure. The simplified analysis ignored several details that are crucial to understanding the behavior of the Beverly Bridge. The manner in which load is applied to the truss is of tremendous importance, and ignoring bending in the arch could lead to an underestimation of its internal stresses. The finite element analysis also showed that the arch served to restrain the central portion of the structure. This led to a reduction of the tensile stresses in the bottom chord and an increase in the top chord compression, compared to the simplified analysis.

5.3 Construction

The final goal of this study is to determine the practicality and cost of rebuilding the Beverly Bridge. If constructed, it would not be the only one of its kind in the United States. Over 800 timber covered bridges are still standing, and there are over 20 of similar length and construction that are currently carrying traffic. All are in rural areas, and it is not clear how many of them have a load restriction.

The Smith Millennium Bridge serves as a possible precedent for the rebuilding of Beverly. It was destroyed by arson but rebuilt to meet HS-20 loading. Though built with glulam rather than sawn lumber, traditional details were used in order to match the original construction.

This example shows that it is possible to build a covered bridge that serves as a tribute to a historical structure and a fully functional piece of the community.

Cost of construction was also considered. It would be impossible to obtain an exact cost estimate because many details of the bridge construction are unknown and it is unclear how much work would be required at the abutments. JR Graton of 3G Construction, the preeminent covered bridge builders and renovators today, estimates that it would cost between 1.5 and 2 million dollars to rebuild the bridge today. Comparison with the cost of the Smith Millennium Bridge reinforces this rough estimate. This cost is within the range of similar span steel and concrete bridges built in recent years, according to construction estimate values from the New York State DOT, but the estimate is higher than the average cost for similar bridges. For past bridge reconstruction and rehabilitation projects, timbers have been donated by local forestry services (Pierce 2005). This would reduce the cost, and perhaps bring it in line with a typical concrete bridge replacement.

It would be both possible and beneficial to the community to rebuild the Beverly Bridge. The analysis in Chapter 3 led to the conclusion that the arch would be overstressed under HS-15 loading, but further study could contradict this and the use of a timber species other than yellow poplar could bring stresses within allowable limits without sacrificing historical authenticity of the structural form. Also, loading on the bridge could be limited, as is the case with timber covered bridges with similar spans nationwide. The cost of reconstruction is estimated to be within the range of comparable modern steel and concrete bridges, meaning that a reproduction of Chenoweth's Beverly Bridge would be a viable option for the Tygarts Valley River crossing in Beverly. Covered bridges are known to attract visitors, and Chenoweth's Philippi Bridge is one of the most visited attractions in the state of West Virginia. The bridge would serve as homage to its builder, especially since it would be located directly adjacent to the Lemuel Chenoweth House and Museum. A third Beverly Covered bridge would undoubtedly be able to serve its local community in both form and function for decades to come.

5.4 Future Work

This study was intended to lay the groundwork for future study of the technology of Lemuel Chenoweth and the Beverly Bridge. Kemp at West Virginia University has studied both of Chenoweth's standing bridges at Barrackville and Philippi. Randy Allan has used letters and historical records to tell the story of Chenoweth's life. The next step would be a comprehensive study of the construction and technology of Chenoweth's structures. In addition to the bridges, he has also worked on churches and houses, including his own home in Beverly. A thorough study of his structures would be useful to understanding one of the last and most prolific covered bridge builders, as his structures were completed as iron bridges were becoming more common. He worked on four structures with spans over 100 feet that were standing well into the twentieth century. His technology may even be useful in the construction of new long span timber trusses for buildings and bridges.

Historical research should continue to be conducted in order to shed light on the modern history or Chenoweth's bridges and his past influences. If Lewis Wernwag was involved in the construction of Beverly, there should be documentation of this in bridge contracts located in the Library of Virginia, in Richmond. Archives at West Virginia University also have newspaper from Randolph County in the early 1950s on microfilm. Extensive research of these records should reveal more information about the decision to demolish the bridge and documentation of the event. This thesis was focused primarily on technical details of the Beverly Bridge, but broader details of the bridge's history would only serve to reinforce its cultural importance.

The results from this thesis showed that reconstruction of the Beverly Bridge is feasible, but it must be studied more closely before this can happen. Laboratory testing of typical Chenoweth connections and splices would be needed in order to build an accurate analytical model. A full three dimensional model of the bridge would also be useful in studying the bridge's performance under combined loading. The floor system was also neglected for this thesis, but it must be carefully designed to resist wheel loads. Additionally, long term deflections of the timber structure and possible settlement of the abutments should be studied.

Applications of this study to the broader construction field are also evident. Timber structures are known to typically have lower embodied carbon emissions than their steel or concrete equivalent. An analysis of the environmental impact of a covered bridge at Beverly compared to its steel girder replacement could provide additional justifications for its reconstruction, as well as the construction of new timber bridges. Ultimately, cost of construction should be balanced with a structure's cultural and environmental impact in order to evaluate its benefits.

Appendix A: Locations of Chenoweth's Bridges

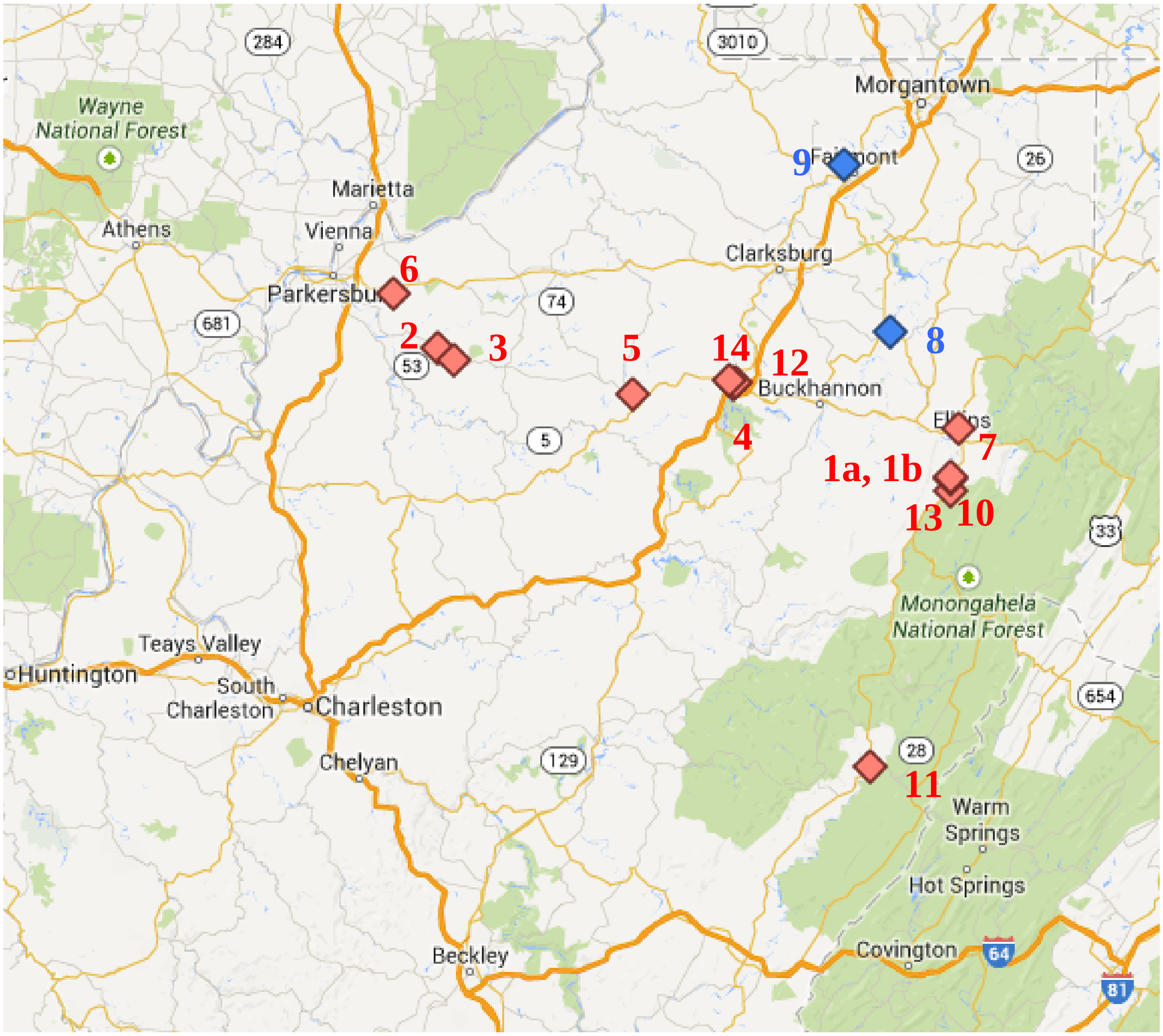

Figure A 1: Chenoweth Bridge Locations, relative to current towns and roads. Note that most are on what was the Staunton-Parkersburg turnpike between Beverly (Bridge numbers 1, 13, and 10) and Parkersburg (Bridge number 6). Blue are existing, red are demolished. Legend below. Map from Google Maps

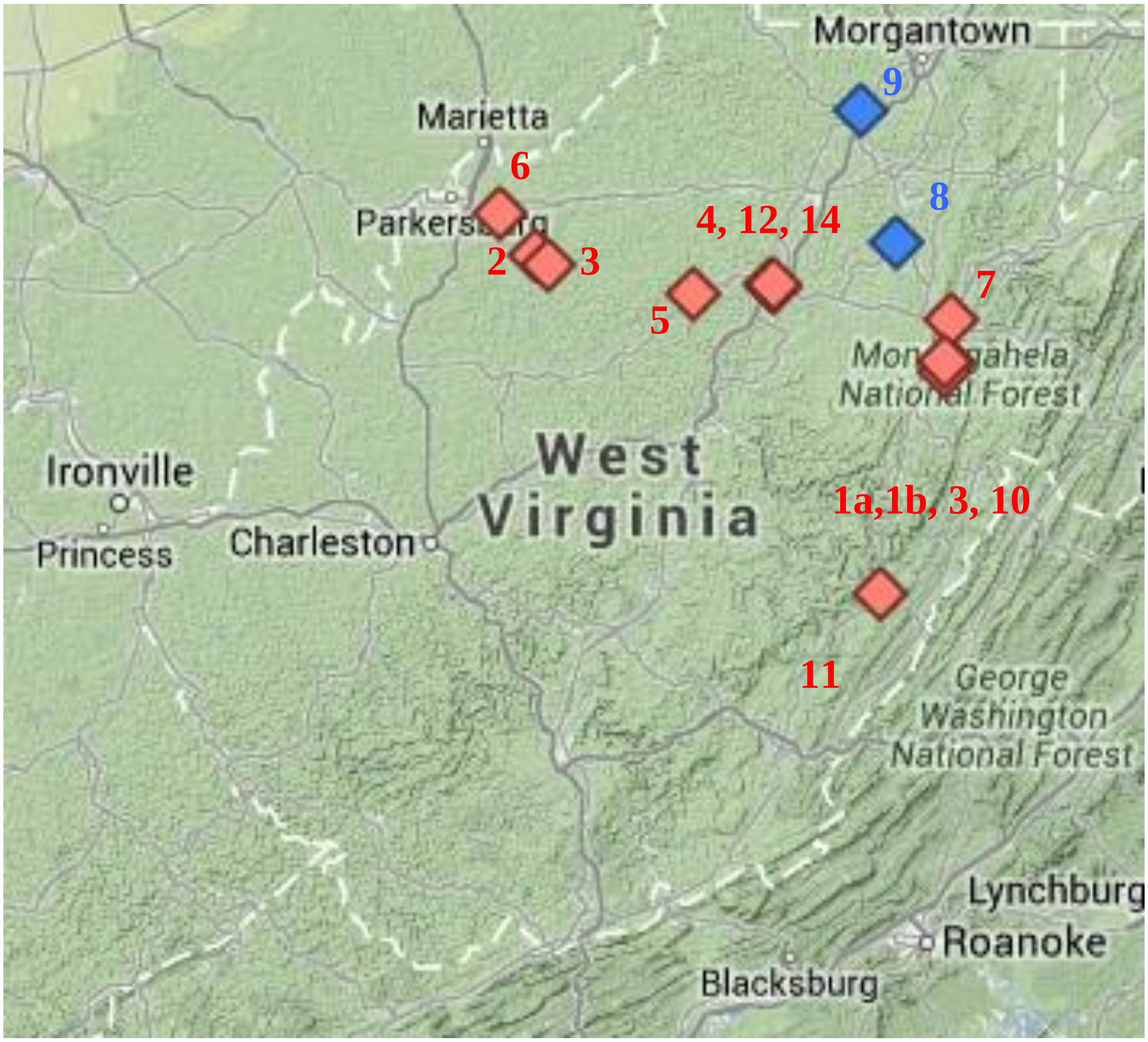

Figure A 2: Locations of Chenoweth's Bridges within the context of West Virginia's borders and terrain. Map from Google Maps.

Table A.1

#	Name	Description	Status
1a	Beverly	1847, 154 feet	Destroyed during Civil War
1b	Beverly	1873, 154 feet	Demolished
2	Hughes River North Fork	1848	Destroyed
3	Hughes River South Fork	1848	Destroyed
4	Middle Fork River Bridge	1848	Destroyed
5	Fink Creek	1848, 93 feet (not covered)	Destroyed
6	Stillwell Creek	1849, 56 feet	Destroyed
7	Leading Creek	1850, 66 feet	Destroyed
8	Philippi	1853, 312 feet	Standing, active
9	Barrackville	1853, 144 feet	Standing, Pedestrian only
10	Stalnaker	1855	Destroyed
11	Marlinton	1855, 300 feet	Destroyed
12	West Fork River		Destroyed
13	Files Creek Bridge	1858, 60 feet	Destroyed
14	Polk Creek	1858	Destroyed

Appendix B: Supplementary Photographs

All Photographs were taken by the author in West Virginia during January, 2014

Figure A 3: Current bridge at Beverly Bridge Location

Figure A 4: West Abutment of the Beverly Bridge, mostly obscured by soil and new concrete

Figure A 5: Modern Beverly Bridge, view from East bank of Tygarts Valley River

Figure A 6: Construction of the Beverly Bridge, photo courtesy of Randy Allan

Figure A 7: Beverly Bridge Immediately Prior to Demolition, photo courtesy of Randy Allan


Figure A 8: Salvaged Beverly Bridge Timbers


Figure A 9: Salvaged Beverly Bridge Timbers

Figure A 10: Salvaged Beverly Bridge Timbers

Figure A 11: Salvaged Beverly Bridge Timbers

Figure A 12: Salvaged Beverly Bridge Timbers. Note the cut area and single bolt hole of the supporting member.

Figure A 13: Salvaged Beverly Bridge Timbers. Note the large transverse cut area, likely the location of an arch and bottom chord intersection

Figure A 14: Salvaged Beverly Bridge Timbers

Figure A 15: Salvaged Beverly Bridge Timbers, showing growth rings of the member


Figure A 16: Salvaged Beverly Bridge Timbers, showing axe marks


Figure A 17: Salvaged Beverly Bridge Timbers

Figure A 18: Tygarts Valley Presbyterian Church, constructed by Lemuel Chenoweth in 1883


Figure A 19: Roof Structure of Tygarts Valley Presbyterian Church

Figure A 20: Exterior and Steeple of Tygarts Valley Presbyterian Church

Figure A 21: Arch Landing of Barrackville Bridge

Figure A 22: Bottom Chord Splice of Barrackville Bridge

Figure A 23: Bottom Truss Connection Detail, Barrackville Bridge

Figure A 24: Top Truss Connection Detail, Barrackville Bridge

Figure A 25: Half-Lap Arch Splice, Barrackville Bridge

Figure A 26: Barrackville Bridge lateral bracing scheme


Figure A 27: Barrackville Bridge Bracing Support Detail


Figure A 28: Arch Landings and Abutments, Philippi Bridge

Figure A 29: Interior Long Truss framing, Philippi Bridge

Figure A 30: Diagonal Intersection Detail. A similar connection would be used between the two diagonal elements of the panel resting on the abutments at Beverly.

Figure A 31: Philippi Bridge Portal, showing the concrete deck and guardrails

Appendix C: Finite Element Model Data

Table A.2: Joint Locations

Joint	X (in)	Y (in)	Z (in)
1	-43.322	-22.579	72
2	37.678	-22.579	72
3	37.678	-22.579	224
4	-43.322	-22.579	224
5	-165.322	-22.579	72
6	-165.322	-22.579	224
7	-287.322	-22.579	72
8	-287.322	-22.579	224
9	-409.322	-22.579	72
10	-409.322	-22.579	224
11	-531.322	-22.579	72
12	-531.322	-22.579	224
13	-653.322	-22.579	72
14	-653.322	-22.579	224
15	-775.322	-22.579	72
16	-775.322	-22.579	224
17	-897.322	-22.579	72
18	-897.322	-22.579	224
19	-43.322	-22.579	0
20	-165.322	-22.579	42
21	-287.322	-22.579	78
22	-409.322	-22.579	110
23	-449.053	-22.579	121.5
24	-531.322	-22.579	135
25	-589.558	-22.579	144.556
26	-653.322	-22.579	150.5
27	-721.424	-22.579	156.848
28	-775.322	-22.579	157.5
29	-844.62	-22.579	158.338
30	-897.322	-22.579	159.5
31	-1751.322	-22.579	72
32	-1832.322	-22.579	72
33	-1832.322	-22.579	224
34	-1751.322	-22.579	224

35	-1629.322	-22.579	72
36	-1629.322	-22.579	224
37	-1507.322	-22.579	72
38	-1507.322	-22.579	224
39	-1385.322	-22.579	72
40	-1385.322	-22.579	224
41	-1263.322	-22.579	72
42	-1263.322	-22.579	224
43	-1141.322	-22.579	72
44	-1141.322	-22.579	224
45	-1019.322	-22.579	72
46	-1019.322	-22.579	224
47	-1751.322	-22.579	0
48	-1629.322	-22.579	42
49	-1507.322	-22.579	78
50	-1385.322	-22.579	110
51	-1345.592	-22.579	121.5
52	-1263.322	-22.579	135
53	-1205.086	-22.579	144.556
54	-1141.322	-22.579	150.5
55	-1073.22	-22.579	156.848
56	-1019.322	-22.579	157.5
57	-950.025	-22.579	158.338
58	-1529.391	-22.579	71.427
61	-266.708	-22.579	72.169

Table A.3: Member Connectivity

Frame	Joint 1	Joint 2	Length (in)	Member Type
1	1	2	81	BOTTOM
2	2	3	152	VERTICAL
3	3	4	81	TOP
4	4	2	172.235	DIAG2
5	3	1	172.235	DIAG2
6	4	1	152	VERTICAL
7	1	5	122	BOTTOM
8	5	6	152	VERTICAL
9	6	4	122	TOP
10	6	1	194.905	DIAG1
12	37	58	22.076	BOTTOM
13	8	6	122	TOP
14	8	5	194.905	DIAG1
15	7	9	122	BOTTOM
16	37	40	194.905	DIAG1
17	10	8	122	TOP
18	40	50	114	VERTICAL
19	9	11	122	BOTTOM
20	50	39	38	VERTICAL
21	12	10	122	TOP
22	39	51	63.472	DIAG1
23	11	13	122	BOTTOM
24	51	42	131.433	DIAG1
25	14	12	122	TOP
26	42	52	89	VERTICAL
27	13	15	122	BOTTOM
28	52	41	63	VERTICAL
29	16	14	122	TOP
30	41	53	93.037	DIAG1
31	15	17	122	BOTTOM
32	53	44	101.868	DIAG1
33	18	16	122	TOP
34	44	54	73.5	VERTICAL
35	19	20	129.027	ARCH
37	21	22	126.127	ARCH
38	22	23	41.361	ARCH
39	23	24	83.37	ARCH
40	24	25	59.015	ARCH

41	25	26	64.041	ARCH
42	26	27	68.397	ARCH
43	27	28	53.902	ARCH
44	28	29	69.303	ARCH
45	29	30	52.715	ARCH
46	31	32	81	BOTTOM
47	32	33	152	VERTICAL
48	33	34	81	TOP
49	34	32	172.235	DIAG2
50	33	31	172.235	DIAG2
51	34	31	152	VERTICAL
52	31	35	122	BOTTOM
53	35	36	152	VERTICAL
54	36	34	122	TOP
55	36	31	194.905	DIAG1
56	58	35	99.933	BOTTOM
57	54	43	78.5	VERTICAL
58	38	36	122	TOP
59	38	35	194.905	DIAG1
60	37	39	122	BOTTOM
61	43	55	108.798	DIAG1
62	40	38	122	TOP
63	55	46	86.107	DIAG1
64	39	41	122	BOTTOM
65	46	56	66.5	VERTICAL
66	42	40	122	TOP
67	56	45	85.5	VERTICAL
68	41	43	122	BOTTOM
69	45	57	110.709	DIAG1
70	44	42	122	TOP
71	57	18	84.196	DIAG1
72	43	45	122	BOTTOM
73	18	30	64.5	VERTICAL
74	46	44	122	TOP
75	30	17	87.5	VERTICAL
76	45	17	122	BOTTOM
77	49	58	23.027	ARCH
78	18	46	122	TOP
79	18	29	84.196	DIAG1
80	47	48	129.027	ARCH
81	58	48	104.174	ARCH

82	49	50	126.127	ARCH
83	50	51	41.361	ARCH
84	51	52	83.37	ARCH
85	52	53	59.015	ARCH
86	53	54	64.041	ARCH
87	54	55	68.397	ARCH
88	55	56	53.902	ARCH
89	56	57	69.303	ARCH
90	57	30	52.715	ARCH
91	29	15	110.709	DIAG1
92	15	28	85.5	VERTICAL
93	28	16	66.5	VERTICAL
94	16	27	86.107	DIAG1
95	27	13	108.798	DIAG1
96	13	26	78.5	VERTICAL
97	26	14	73.5	VERTICAL
98	14	25	101.868	DIAG1
99	25	11	93.037	DIAG1
100	11	24	63	VERTICAL
101	24	12	89	VERTICAL
102	12	23	131.433	DIAG1
103	23	9	63.472	DIAG1
104	9	22	38	VERTICAL
105	22	10	114	VERTICAL
108	10	7	194.905	DIAG1
109	7	21	6	VERTICAL
110	21	8	146	VERTICAL
111	37	49	6	VERTICAL
112	49	38	146	VERTICAL
113	7	61	20.615	BOTTOM
114	35	48	30	VERTICAL
115	5	20	30	VERTICAL
116	61	5	101.385	BOTTOM
117	21	61	21.423	ARCH
118	61	20	105.779	ARCH

Table A.4: Member Section Assignments

Section Name	Material	Width (in)	Height (in)
ARCH	Poplar	14	14
BOTTOM	Poplar	16	18
DIAG1	Oak	8	8
DIAG2	Oak	6	6
TOP	Poplar	10	12
VERTICAL	Poplar	12.75	8

Appendix D: Existing Long Span Burr Truss Bridges

Name	State	Spans	Length (feet)	Year	Status
Medora or Dark	Indiana	3	434	1875	pedestrian as of 1972
Deer's Mill	Indiana	2	275	1878	closed in 1968
Conley's Ford	Indiana	1	192	1907	open to traffic
Bridgeton	Indiana	2	267	2006	replaced 1868 bridge
Nevins	Indiana	1	155	1920	
Thorpe Ford	Indiana	1	163	1912	closed in 1961
Mecca	Indiana	1	150	1873	Closed in 1964
Crooks	Indiana	1	132	1855	Rebuilt 1967
State Sanatorium	Indiana	1	148	1912	Closed, private
Huffman Mill	Indiana	1	136	1864	Bypassed 2003
Oakalla or Shoppell	Indiana	1	152	1898	open to traffic
Forsythe Mill	Indiana	1	198	1888	open to traffic
Norris Ford	Indiana	1	170	1916	open to traffic
Cedar Ford	Indiana	1	141	1885	in storage
Newport	Indiana	1	180	1885	closed
Eugene	Indiana	1	192	1873 or 1885	restored 1995, open to pedestrians only
Blair	New Hampshire	2	293	1869	temporarily closed to traffic
Smith Millennium	New Hampshire	1	167	2001	original bridge destroyed in 1993, open to traffic
Rowell	New Hampshire	1	167	1853	rebuilt 1996, open to traffic

Perrin	New York	1	154	1844	closed to vehicles, restored 1969 and 1993
Herline or Kinton	Pennsylvania	1	136	1997	rebuilt 1902 bridge, open to traffic
Pleasantville	Pennsylvania	1	139	1852	open to traffic
Greisemer's mill	Pennsylvania	1	140	1868	
Wertz or Red	Pennsylvania	1	218	1867	closed
Dreibelbis Station	Pennsylvania	1	189	1869	open to traffic
Stillwater	Pennsylvania	1	151	1849	closed
Rupert	Pennsylvania	1	185	1847	open to traffic
Academia or Pomeroy	Pennsylvania	2	278	1902	restored 2009, pedestrians only
Pinetown Road	Pennsylvania	1	135	1867	open to traffic
Hunsecker's Mill	Pennsylvania	1	181	1975	open to traffic
Jackson's Mill	Pennsylvania	1	143	1985	open to traffic
Colemanville Mill	Pennsylvania	1	160	1990	open to traffic
Banks	Pennsylvania	1	134	1889	open to traffic
Bogert's	Pennsylvania	1	172	1841	closed
Rex's	Pennsylvania	1	138	1858	
Adair's or Cisna Mill	Pennsylvania	1	160	2007	rebuilt 1864 bridge, open to traffic
Rice or Landisburg	Pennsylvania	1	132	1869	rebuilt 2003, open to traffic
Forksville	Pennsylvania	1	163	1850	open to traffic
Cambridge Village or Museum	Vermont	1	163	1845	modified 1951, double barrel, closed to traffic

Poland or Station or Cambridge Junction	Vermont	1	153	restored 2004	open to traffic
Meem's Bottom	Virginia	1	203.5	1894	open to traffic
Philippi	West Virginia	2	304	1852	rebuilt 1991, double barrel, concrete floor
Carrolton	West Virginia	1	156	1855	rebuilt 2002, open to traffic

Appendix E: Material Unit Weights for Analysis

	Item	Unit Weight	Unit	Weight Per Truss Panel (Pounds)
Floor	primary beam	0.6	psf	54
	secondary beams	2.9	psf	264
	platform	10.1	psf	928
Truss	diagonals	19.6	lb/ft	319
	top chord	25.6	lb/ft	260
	bottom chord	61.3	lb/ft	624
	vertical	21.7	lb/ft	275
Bracing	struts	7.7	lb/ft	15
	primary lateral	10.4	lb/ft	94
	braces	8.9	lb/ft	243
	peak support	1.2	psf	108
Cover	siding	6.4	psf	818
	shingles	2.5	psf	324

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