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Structural Rehabilitation of a Historic Covered Bridge: Bridgeport Covered Bridge

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Abstract

The Bridgeport Covered Bridge was constructed in 1862 by David Inglefield Wood of the Virginia Turnpike Company as a major toll road to support the development of the Comstock Silver Lode across the Sierra Nevada Mountains in western California. The bridge has a substantial span of 63.4 meters (208 feet) over the South Fork of the Yuba River in Nevada County, California. It utilizes a Howe Truss fitted with a supplementary timber arch chord on either side of each main truss. The span has the distinction of being the longest remaining single-span covered bridge in the United States.

After providing a temporary structural stabilization design in 2013, Buehler began the full structural rehabilitation design of the bridge which both maintained the historical nature and detailing of the bridge while also making discrete structural improvements to allow the bridge to continue to be a historical resource for the State of California for many years to come.

1 Introduction

The resiliency of architecture and structure are exemplified by the historic style and nature of a wood covered bridge. Throughout the United States, the expansion of settlers moving west in the 1800s necessitated the development of roads and rail links to connect people and supply chains to the areas of commerce and development. No stranger to these endeavors were the bridge builders of that time who coupled the experience and ingenuity of predecessor projects, civil engineers, and patents, domestic and abroad, to create these vital links built to stand the test of time.

This paper is intended to highlight the history of the Bridgeport Bridge construction and document the techniques utilized to rehabilitate, tune and test a historic covered bridge. Through, and because of, the rehabilitation, much of the historical fabric has remained intact and will be on display for generations to come.

The bridge has a vibrant history of service which has endured many twists and turns in recent years. Design was completed in 2018, construction commenced in 2019, and in the fall of 2021, the bridge was re-opened to the public.

History of the Bridgeport Bridge

Following the discovery of silver in Western Nevada in 1859, the Bridgeport Bridge was constructed along a new toll road in order to provide a safe and direct passage through a portion of the Sierra Nevada mountains towards Sacramento and San Francisco. The bridge was built by David Inglefield Wood in 1862.

Following the peak of the silver mining in 1876, the bridge remained in private ownership until 1901, when it was declared a route along a public highway. The bridge was then utilized for automobile traffic until 1973, when a new concrete span was constructed upstream. In 1986, California State Parks acquired the bridge and adjacent property to develop the South Yuba State Park.

In January 1997, a flood damaged a significant portion of the bridge and was restored later that year. In 2013, the bridge was closed to pedestrian use until it was temporarily stabilized in 2015 and design efforts for the rehabilitation began shortly thereafter.

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Components of the Bridgeport Covered Bridge

The Bridgeport Bridge is comprised of timber and iron with stone masonry abutments. The primary bridge structure utilizes a truss on either side of the floor deck which are flanked with a supplemental, segmented arch on the inside and outside face of each truss. The truss has an upper and lower chord with double braces and single counter-braces within the web. The vertical web members consist of two iron tension rods. The chords, braces, counter-braces and tension rods are coincident at cast iron nodes.

Material Investigation

In order to determine the strength of existing materials, material testing and investigation was required during design. The existing structure was assessed for conditions of decay and damage. Most of the primary members observed appeared to be from the original construction.

The original timbers used to construct the bridge were old-growth timber likely sourced very near to the bridge site (Miller & Knapp, 2013). Wood members were visually graded to obtain member allowable stresses. Specific areas of core decay were identified in the top chord along with local crushing at tension rod connections. One tension rod had failed near the top threaded portion of the rod just below the nut. The existing tension rods were tested for strength and microstructure and the wrought iron rod strength was found to be consistent with historic material strengths.

Rehabilitation Design Criteria

The Bridgeport Bridge is located in the foothills of western Nevada County and spans across the southern fork of the Yuba River. Numerous faults have been identified within sixty miles of the site and seismicity is considered “high”. The bridge, being a California Historical Landmark, was designed to the 2013 California Historic Building Code and ASCE 7-10.

No original construction documents or calculations have been found. Early photographs of the bridge show signage limiting the maximum bridge loading to three tons. Early engineering manuals (Engineering Field Manual, 1917) make note that “the maximum uniformly distributed live load will not exceed the maximum uniformly distributed dead load.” A design live load of sixty-five pounds per square foot was used which correlates with historic engineering manuals while also minimizing the degree of member augment or replacement due to gravity load overstress. In preliminary strength assessment studies, the wrought iron tension rods were found to be the weak link (Gasparini, et al., 2016). Early judgments were made during design that many of the tension rods required replacement in order to first improve the structural resiliency of the bridge, and also to maximize the strength and load carrying capacity of the remaining bridge members.

Lateral loads were applied to the bridge per the California Historic Building Code which allows for a twenty-five percent reduction in lateral forces from the current Building Code. Wind and seismic loads were obtained from ASCE 7 and compared to AASHTO loading.

Analytical Modeling

Various tools were used to study and analyze the bridge. The primary structural analysis modeling program used was SAP Ultimate. The bridge members and their connections were modeled to appropriately assign tension-only (iron tension rods) or compression-only members (arch, braces, and counter-braces) to properly simulate the anisotropic members and connections. The analytical model also provided a way to study various loading and construction sequences. Variations were studied which engaged the arch chords at different sequences relative to the release of the shoring for dead loading and introduction of live loading. In the different sequences studied, the arch attracts more load the earlier that it is engaged. Conversely, the truss action (as demonstrated by the upper chord and tension rod demand) diminishes with early arch engagement.

Truss System Description

The Bridgeport Covered Bridge trusses consist of two Howe trusses which are approximately 16’-6” (5.03m) between laminated chord centroids. The components of the truss include horizontally laminated top and bottom chords consisting of eight laminations of approximately 2” (51mm) nominal x 20’-0” (6.10m) DF-L sawn lumber stitched together with bolts and spiked nails. Laminated planks were likely

used in lieu of hewn timbers for economy's sake (Bennett, 2002). The web members consist of two 4-3/4"x 10-1/4" (121mm x 260mm) braces with a single 4-3/4" x 7-3/4" (121mm x 197mm) counter-brace opposing. Two steel tension rods occur at each bay. The work point of the tension rods, strut, counter-strut, and top or bottom chord coincide with a nodal casting. At the abutments, a 12"x16" (305mm x 406mm) bolster beam is located which extends from the abutment out past the location where the arch chord passes the bottom truss chord.

Arch System Description

The arch consists of two 4-3/4" x 13" (121mm x 330mm) chords which abut both the inside face and exterior face of the truss. The arch chords are bolted to each double truss brace with two ¾" (19mm) diameter bolts with blocking between the brace and counter-brace at each bolt attachment.

Which System is Dominant?

From the outset, the most critical aspect of the bridge analysis was understanding the load path and then appropriately modeling and analyzing the structure. With two capable gravity support structural systems (Howe truss and arch), the rehabilitation must honor the original load paths and load sharing so as to minimize new strengthening elements and new stresses in the historic structure. In order to understand the load path, Buehler aimed to consider the original designer's intentions through studying the bridge member's proportions, material strengths, and available historic references.

While there are many bridges around the world built with a combination truss arch system, historical practice and modern interpretations seem to show that there is not uniformity of thought or practice on how these structures are to perform. Across many of the truss-arch combination structures, the arch system is typically proportionally much stiffer than the truss system. German engineer Karl Culmann noted the relative arch stiffness to truss stiffness during his American tour in the mid-nineteenth century (Rinke, 2016). While the stiffness difference between the two systems may suggest a more dominant arch participation, other references suggest the arch has been considered supplementary to the truss. JJ Daniels, a bridge builder who built Long truss bridges (a predecessor patent to the Howe truss), would attach the arch to the truss after falsework removal. (Barker, 2015). Initially, the truss would support the bridge self-weight, with the arch taking the subsequent live loads after engagement.

The Bridgeport Bridge arch is attached to the truss at each strut and as it passes across the bottom chord. Different than a Long truss in that the vertical tension members are slender iron; the truss web members cannot directly apply load to the arch through a vertical member congruent with the load direction. Therefore, a majority of the load transfer into the arch occurs at the connection of the arch to the bottom chord and bolster beam. A study of the bridge structure was performed to review the proportions of the structural members relative to loading. Various historic engineering documents (including Modern American Bridge Building (Tower, 1874)) provide design criteria and member capacity for trussed bridges. While published following the Bridgeport Bridge's erection, it is fair to assume limited changes to design methodology in that time span. Backwards engineering of the upper chord, end brace, and end vertical rod shows very close alignment of member proportion to allowable wood stresses allowed in the 1874 design manual. The 1874 reference provided a 15 ksi (103 MPa) working stress for iron rods; which the study showed was exceeded with a 21 ksi (145 MPa) working stress under dead loads without arch contribution.

The high stress within the end tension rods begs the question of what early value engineering decisions may have been made. It is noted that a 1-1/2" (38mm) square rod would produce a 15 ksi (103 MPa) dead load stress which would correspond to the working stress limit noted in the 1874 publication. Another hypothesis is that the apparent undersized end rods may be attributed to a reduction in end rod stress due to the strength and stiffness increase in this portion of the span from the bolster beam. The 12x16 (305mm x 406mm) bolster beam provides an increased flexural capacity of the bottom chord and detracts from the truss action of the truss assembly. A cursory study model showed that the presence of this bolster beam reduces the forces in the web members by approximately fifteen percent. With this assumption, tension rod stresses would be approximately 18 ksi (124 MPa) which offers better alignment with historic rod capacity. It is unclear how or if the original designers would have estimated the degree to which the bolster beam would decrease truss action.

In addition, construction sequencing studies showed that engaging the arch earlier in the erection sequence would increase arch stress due to the difference in arch stiffness relative to the truss. When the arch is

initially engaged for both dead and live loads, it was found that the arch stress was almost two times higher than when the arch engagement was postponed until live load introduction. This showed that if the Arch chord was assumed to be the primary system, it would be overstressed for historic and modern capacity values without significant strengthening.

Bridge Improvements

Various improvements to the bridge were made in the rehabilitation design. In general, most repairs consisted of the replacement of damaged timber in kind. Truss repairs consisted of complete replacement of the top chord and selective replacement of damaged bottom chord laminations. Improvements were made to the stitching of the laminations with nail spikes, timber screws and stitch bolts. Hidden timber screws were used locally at the midspan where additional lamination displacement was noted and tension stresses were high. Damaged portions of the arch were replaced in kind. The arch bearing hardware appeared to have been incorrectly installed in the 1971 repairs which resulted in a shortened arch span at the south arch. The rehabilitation corrected this condition and provided adjustable arch bearing seats to allow for future arch tuning to increase or decrease force within the arch. Repairs and replacements to the floor decking and exterior siding and roofing were also made. Improvements to the Tension Rods and Abutments are discussed further below.

Tension Rods

The Secretary of the Interior Standards for Rehabilitation (National Park Services) specify that "deteriorated historical features will be repaired rather than replaced. Replacement of missing features will be substantiated by documentary and physical evidence." Results from the structural analysis indicated that the rods were significantly overstressed. Due to the age and condition of the wrought iron rods and the high number of loading fatigue cycles over time, replacement of the existing highly stressed wrought iron rods was recommended.

Iron rods manufactured in the 1850s time period had variable strength characteristics due to archaic manufacturing and quality control issues. (In addition, during the rehabilitation, stamps observed on the rods showed seventeen different foundry identification stamps from foundries all over the world.) Iron is considered a brittle material due to the relatively small elongation of the material prior to tension failure compared to steel. This characteristic is not desirable from a performance or safety standpoint. ASTM A588 grade 50 (345 MPa) weathering steel was considered for replacement of the existing rod material. In addition to strength, the weathering steel was evaluated for texture and finish similarity and found to be the best option due to strength, source availability, and natural corrosion resistance characteristics.

Abutments

The original abutments from 1862 were dry stacked granite blocks resting on a rock outcropping on the north and river cobbles and boulders on the south end. In 1971, the original bridge abutment was raised thirty inches (762 mm) based on documents by Gillett Harris Duranceau Associates Sheets 1-9 (stamped preliminary) dated July 29, 1971. Extensive soil and abutment studies to evaluate the support conditions determined that the existing gravity rock walls were vulnerable to failure due to gravity, soil, lateral, or seismic loads.

New grouted piles and pile caps with abutment walls were placed within the footprint of the existing rock walls. The rock walls were drilled and epoxy doweled to the abutment walls to act as a veneer. Rock joints were cleaned and fully mortared to correct voids.

Tuning

The performance of the Howe truss over time is reliant upon regular monitoring and occasional tightening of the tension rods. Over time, various environmental factors will contribute to the reduction of tensile forces within the tension rods after the initial prestress state. The tension rods pass through the top chord, the horizontal strut perpendicular to the span of the bridge, cast iron nodes (top and bottom) and the bottom chord. Steel plate washers are located above and below the top and bottom chords, respectively, which bear against the chords. After the initial prestress, member seating, the timber volumetric shrinkage losses, and expansion and contraction of the wood and steel members due to temperature changes will all affect the prestress. Most of the prestress loss is expected to occur within the first few months after the initial stressing.

(Gasparini, et al., 2020). After these months, the truss tension rods will be examined and additional tensioning will be performed.

The initial tuning sequence for the Bridgeport Bridge was developed by Tim Andrews of Barns and Bridges of New England and Buehler. In this sequence, the truss was rebuilt in place while suspended from the shoring truss. While suspended, the vertical tension rods are initially tightened finger tight plus half turn. The struts and counter struts are then shimmed or shortened to ensure firm bearing. An epoxy grout was used to fill voids at the cast iron bearing shoe to provide a tight fit. After firm bearing was established, the vertical rods were tightened to a "full effort of a man" using an eighteen inch (457mm) long wrench. Vertical rod pairs were checked for equal tension and diagonals for equal bearing by hammer sounding the members.

Following the initial tuning, the arch chords were replaced and the jacks supporting the bridge structure were lowered to engage the truss and arch. At this point, the structure supported the full dead load of the bridge. During this sequence, camber and deflections were carefully monitored and recorded. During the construction process, continued conversations were had regarding the arch loading sequence. It was determined that allowing the arch to partially engage at the dead loading conditions would help restrict deflection losses while having minimal effect long-term member stresses and connection forces. Load cell monitoring at the arch bearing seats was used to maintain and monitor the arch reaction loads. The bearing seats were adjusted to maintain symmetric loading and keep loading within acceptable tolerance as the full dead load and proof live load on the bridge was introduced. Due to the shoring truss location, elements of the bridge were not able to be re-built until the shoring truss was removed. In order to properly simulate dead loads of the bridge, simulated loads utilizing hung water totes were used. After simulated truck passage proof load, the shoring truss was removed and the remaining bridge stringers, siding, and roofing were installed.

During tuning, a selection of the vertical tension rods and the arch chord members at the buttress bearing seat were instrumented to measure forces in members. Strains in the tension rods, harmonics in the tension rods, force on the arch bearing seat, and strain in the truss bottom chord were recorded at various stages of construction and loading. This data will be helpful in future retuning efforts to evaluate loss of prestress and determine the level of adjustment each rod may require. In future retuning efforts, the arch chords may be utilized to relieve the self-weight loading on the trusses. With this relief, the tension rods may be tightened and creep and camber losses may be able to be re-established.

It must be noted that prestress losses due to the tension rod bearing on the wood top and bottom chords has been a point of refinement within the history of wooden covered bridge construction. The original bearing seat design provided a flat or dimpled surface where it met the truss chords. The rod forces were transferred through the timber chords in compression perpendicular to grain. This action affects the tension rod prestress as the wood shrinks, crushes, or expands due to moisture content changes. Amasa Stone, William Howe's brother-in-law and colleague, is recognized for improving the node casting by integrating a sleeve through the timber truss chords allowing for minimal prestress losses due to wood behavior (Barker, et al., 2015). This sleeve also allowed for improved shear transfer between the web members and the chords. The Bridgeport Bridge node castings did not have this integrated sleeve.

Shoring

A conventional external shoring system was envisioned in the construction documents which was modeled after shoring used in the 1971 repairs utilizing steel shoring towers supported in the river gravels. Due to more intensive CEQA (California Environmental Quality Act) permit issues to protect the habitat in the river channel, the contractor elected to pursue a modular steel truss system that was internal to the bridge structure.

This box steel truss, named the Mabey Truss, was assembled at one end and launched (pushed) through the bridge and across shoring towers located on the amended footings of the temporary stabilization system coincident with the arch to bottom chord intersection point of the bridge.

The shoring truss was intended to carry gravity and lateral wind loads during construction. One complication of this shoring system was the inability to support a fully loaded dead weight of the bridge while allowing for completion the bridge floor decking until the shoring rods that supported the work platform which was suspended below the bottom chord were removed. This limitation in the shoring design

was accommodated by field load testing of the bridge prior to shoring removal and completion of construction.

Repair and Re-Assembly

After disassembly, re-assembly began with repair work on the bottom chord damage. Lamina were replaced and stitched together with GRK timber fasteners. The entire bottom chord assembly was then retro-stitch bolted to strengthen for shear transfer in the staggered laminations. Original cast iron stitch bolts were retained for the historic fabric and augmented with supplemental staggered stitch bolts. For minor wood imperfections, the contractor utilized repair epoxy to fill holes and checks and provide uniform support for the cast iron seats on the truss chords.

New bolster beams were added at both abutments and damaged solid sawn chord sections at ends were replaced with step-lapped connections to laminated chord material. Truss re-assembly and exchange of damaged members continued for the truss bracing to provide base geometry for a completely new laminated top chord. The new top chord consisted of twenty-foot-long (6.10m) 2”x16” (51mm x 406mm) boards which were laminated with GRK timber fasteners and stitch bolted to complete the assembly. Original truss top chord bracing and struts were replaced as needed and re- installed. Steel bracing above the top chord was added in preparation for field drilling of tension rod holes and re-assembly of the rod system. End steel portal frames were attached to the steel top chord bracing assembly. Cast iron seat connections at braces were wedged and filled with epoxy for uniform compression support.

Completion of the initial tuning allowed the assembly of the side walls and roof framing at the center portion of the bridge in preparation for final load testing.

Load Testing

A full-scale dead and moving live load testing program was incorporated into the final approval of the bridge. The dead loads from the skin and decking that were in conflict with the shoring system had to be artificially applied using suspended water totes. The live load testing was accomplished by using calibrated jacks at discrete points against the shoring truss system that was now no longer needed as a gravity support.

Load testing was monitored by a survey of points on the structure to note deflections, strain gages to monitor tension in the bottom chord, and load cells at arch chord seats to monitor compression reactions at abutments. Sounding frequencies of tension rods and arch chord were documented in the testing process.

Future Maintenance and Tuning

The new protective skin on the structure significantly increases the resilience of the wood structure from environmental decay and applied fire retardant will also help protect against California wildfires. The truss is expected to experience long-term creep and shrinkage due to drying and settling of connections. The maintenance plan is to check the tightness of the tension rods and to adjust arch support connection at abutments. The testing process has allowed documentation of tension rod and arch chord sounding frequencies to aid in follow-up adjustments.

Conclusions

The rehabilitation of the Bridgeport Bridge provided an opportunity to preserve a piece of history for many future generations to enjoy and reflect on our past. In the analysis of the Bridgeport Bridge 150 years after its original construction, we were able to apply modern analytical tools and engineering understanding to a statically indeterminate and complex structure. This analysis both validated much of the original design as well as allow improvements to be made where the original design did not meet modern code or design requirements.

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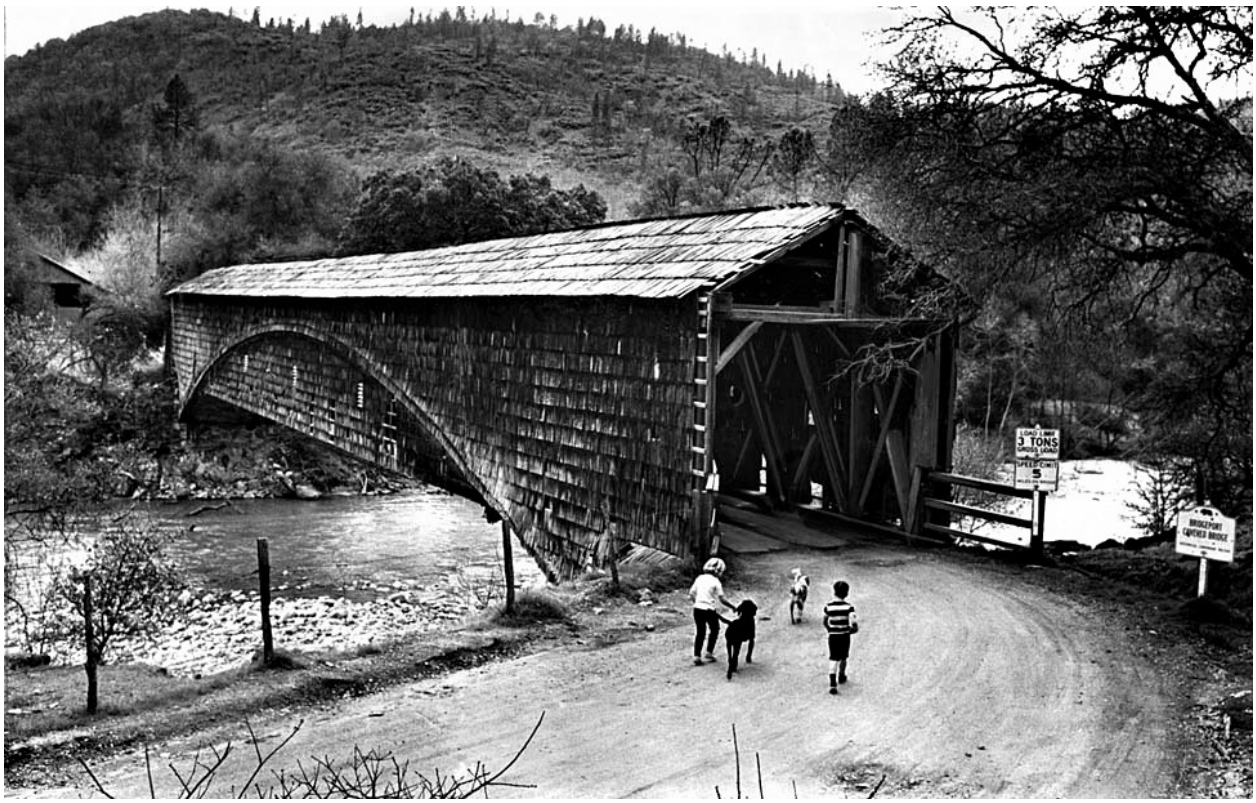

Figure 1 – Bridgeport Bridge circa 1970 (Image courtesy of South Yuba River State Park)

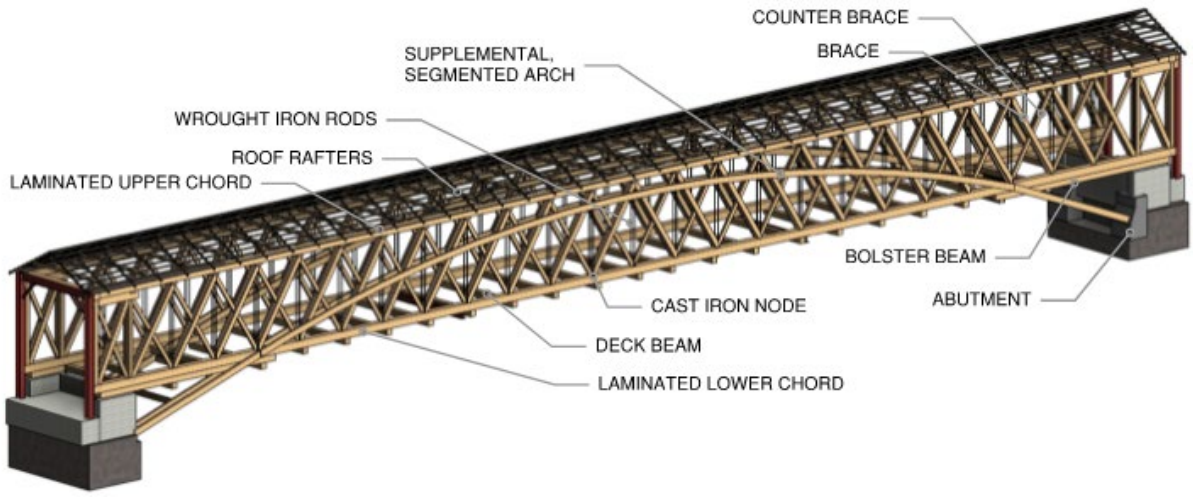

Figure 2 – Graphic of Bridge Components

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Figure 3 – Photograph of Bottom Chord Decay prior to repair (Lawrence Jones)

Figure 4 – Launch of shoring (Lawrence Jones)

Figure 5 – Bare bridge with shoring installed (John Rebenstorff)

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Figure 6 – Load testing bridge prior to shoring removal (Lawrence Jones)

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Figure 7 – Completed Bridge – exterior (Brian Wiens)

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Figure 8 – Completed Bridge – Interior (Brian Wiens)