GILPIN'S FALLS COVERED BRIDGE
Spanning North East Creek at Former (Bypassed) Section of North East Road (SR 272)
North East
Cecil County
Maryland

HAER MD-174
HAER MD-174

PHOTOGRAPHS

REDUCED COPIES OF MEASURED DRAWINGS

FIELD RECORDS

HISTORIC AMERICAN ENGINEERING RECORD
National Park Service
U.S. Department of the Interior
1849 C Street NW
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ADDENDUM TO: HAER MD-174
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WRITTEN HISTORICAL AND DESCRIPTIVE DATA

HISTORIC AMERICAN ENGINEERING RECORD
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GILPIN'S FALLS COVERED BRIDGE

HAER No. MD-174

Location: Spanning North East Creek at bypassed section of North East Road (SR 272), North East, Cecil County, Maryland

Gilpin's Falls Covered Bridge is located at latitude: 39.648611, longitude: -75.955833. The coordinate represents the center of the structure. This coordinate was obtained in July 2009, using a GPS mapping grade unit accurate to +/- 3 meters after differential correction. The coordinate's datum is North American Datum 1983. The Gilpin's Falls Covered Bridge location has no restriction on its release to the public.

Structural Type: Burr truss

Construction Date: 1860

Builder: Joseph G. Johnson, Bay View, Maryland

Owner: Cecil County, Maryland

Original Use: Vehicular bridge; bypassed 1936

Present Use: Historic landmark and tourist attraction

Significance: For almost two centuries, from 1735 until 1926, this site was used for water-powered industries. Erected in 1860 by local bridge builder Joseph G. Johnson, Gilpin's Falls Covered Bridge is a good example of a Burr truss, patented in 1806 and 1817 by Theodore Burr. The Burr truss was used extensively for covered bridges throughout the nineteenth century. After years of decay, the bridge was stabilized and rehabilitated in 2009-10 by timber framer Timothy Andrews, who used traditional timber framing methods, replaced historic fabric with in-kind material, and saved as many original components as possible.

Project Information: The National Covered Bridges Recording Project was undertaken by the Historic American Engineering Record (HAER), a long-range program to document historically significant engineering and industrial works in the United States. HAER is administered by the Heritage Documentation Programs Division (Richard O'Connor, Chief), a division of the National

Park Service, U.S. Department of the Interior. The Federal Highway Administration's National Historic Covered Bridge Preservation Program funded the project.

Christopher H. Marston, HAER Architect, served as project leader. The HAER field team consisted of Anne E. Kidd, field supervisor; Jeremy T. Mauro and Bradley M. Rowley, architects; and Csaba Bartha, ICOMOS intern (Romania). Lola Bennett wrote the history, and Jeremy Mauro wrote the rehabilitation section, in consultation with Tim Andrews of Barns and Bridges of New England. Rachel Sangree wrote the engineering report through an agreement with Johns Hopkins University. David Ames of the University of Delaware produced the large-format photographs. Additional assistance was provided by Jonathan Pohlman of the Cecil County Engineering Department and W. Earl Simmers of the Cecil County Historical Society, as well as Benjamin Schafer of Johns Hopkins University.

Historians: Lola Bennett (history)
Jeremy Mauro (rehabilitation process)
Rachel Sangree (engineering report) with Hannah Blum
2009-2012

CHRONOLOGY

1674	Cecil County formed from part of Baltimore County.
1735	Samuel Gilpin establishes a water-powered sawmill near this site.
1788	Maryland granted statehood.
1805	America's first covered bridge erected at Philadelphia.
1806	Theodore Burr patents Burr truss.
1815	Theodore Burr erects world's largest timber arch bridge across the Susquehanna River.
1831	Joseph G. Johnson born in Cecil County, Maryland.
1860	Johnson erects Gilpin's Falls Covered Bridge.
1900	Johnson dies at Baltimore, Maryland.
1905	William Warburton builds a hydroelectric plant near this site.
1932	Gilpin's Falls Covered Bridge repaired.
1936	Gilpin's Falls Covered Bridge bypassed.
1958	Gilpin's Falls Covered Bridge roof collapses under heavy snowfall.
1959	Gilpin's Falls Covered Bridge repaired by Harry C. Eastburn & Son of Newark, Delaware.
1971	Gilpin's Falls Covered Bridge repaired following several incidents of vandalism.
1989	Maryland Department of Transportation transfers ownership of bridge to Cecil County. Cecil County Historical Society begins fundraising efforts to preserve bridge. Southeast top chord reinforced with steel plates.
1998	Federal Highway Administration launches National Historic Covered Bridge Preservation Program.
2008	Gilpin's Falls Covered Bridge listed in the National Register of Historic Places.
2009	Gilpin's Falls Covered Bridge undergoes extensive rehabilitation by Kinsley Construction Inc. and Barns and Bridges of New England. Historic American Engineering Record documents the structure prior to, and during, rehabilitation.
2010	Gilpin's Falls Covered Bridge rehabilitation project completed. Bridge is reopened to pedestrian use.

MARYLAND COVERED BRIDGES

In 1817, Theodore Burr erected Maryland's first covered bridge, a 4,170' structure across the Susquehanna River at Rock Run.[1] A year later, Lewis Wernwag began construction of the 1,744' Conowingo Bridge across the same river. At least 100 covered bridges have been recorded in Maryland.[2] Over time, ice, floods, accidents, roadway "improvement" projects, arson and neglect took their toll. By 1959, less than two dozen covered bridges were still standing in the state.[3] Today, Maryland is home to six covered bridges, of which four are Burr trusses. At 119', Gilpin's Falls Covered Bridge is the longest covered bridge in the state.

2009 DESCRIPTION OF BRIDGE (PRE-REHABILITATION)

Gilpin's Falls Covered Bridge is a single-span Burr truss covered bridge on stone abutments. The bridge is 119' long (portal to portal) and has a clear span of 99'. The structure is 16'-6" wide between the outer faces of the trusses, with a 13'-wide roadway. The trusses are 20'-0" high, from the top of the top chord to the bottom of the bottom chord. Clearance is 12'-0".

The kingpost trusses have ten structural panels and two shelter panels each. Each truss is composed of a single web of 8" x 10" vertical posts (the center post tapers from 7-$^{3}/_{4}$" x 9-$^{5}/_{8}$" at the neck to 9" x 16" at the base) and 7" x 8" diagonal braces angled down toward the ends of the bridge. The top chords are 6" x 10" planks with a mortise and tenon joint pinned with two 1"-diameter treenails at each post. The bottom chords are two lines of 5" x 10" planks, notched and bolted to the lower ends of the vertical posts.

A pair of timber arches flanks each truss and is notched into the vertical posts and fastened with $^{1}/_{2}$"-diameter bolts. Each arch has two ribs composed of 5" x 10" timbers butted together, end-to-end. The arches spring from below the truss seats at the abutments, rise 13' to the crown and span 99'.

The ends of the bottom chords are seated on bed timbers embedded in the faces of the abutments. The floor system is composed of pairs of 8" x 14" transverse floor beams seated on the bottom chords at each panel point. The outer ends of the floor beams are bolted to the posts. There are ten lines of 4" x 8" stringers on top of the floor beams. The wearing surface is plank decking laid transversely on top of the stringers.

Upper lateral bracing comprises 7" x 7" collar ties notched into the top chord at each panel point and pinned with $^{3}/_{8}$" x 8" wrought-iron spikes. The 4" x 5" cross bracing is notched into the tie beams. Transverse tension rods have been added below the tie beams. There are 2" x 4" knee braces between the vertical post and tie beam at each panel point. Rafters that taper from 2" x 5" at the eaves to 2" x 4" at the ridge and are spaced approximately 2' apart support the gable roof, which is covered with wood shingles fastened to nailers on top of the rafters.

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[1] The Rock Run Bridge stood until 1857.
[2] Richard Sanders Allen, *Covered Bridges of the Middle Atlantic States* (Brattleboro, Vermont: Stephen Greene Press, 1959), 46.
[3] Allen, 107.

Clapboard siding terminating below the eaves covers the bridge. The clapboards are fastened to a series of vertical nailers on the outer faces of the trusses. There is a horizontal, hooded window opening on each side of the bridge at mid-span. The portals have straight, squared openings with clipped corners.

HISTORY

In 1735, Samuel Gilpin harnessed the waterpower at this site to operate a sawmill and a flour mill. Later, a woolen mill occupied the site. There may have been a bridge at this location from an early date, but no records have been found concerning such a structure.

In June 1860, the Cecil County Commissioners approved funds for building several new county bridges, including $2,000 for a bridge over North East Creek at Gilpin's Falls. Three months later, the county placed an advertisement in the local newspaper for proposals for this structure, specifying a 100' "covered Burr arch on a multiple Kingpost truss bridge."[4] On September 11, 1860, the commissioners awarded the bridge contract to Joseph G. Johnson for $2,000.[5] The *Cecil Whig* of December 15, 1860, reported, "the new bridge over North East Creek at Gilpin's Falls [is] in the process of completion."[6] Cecil County Commissioners records indicate that Joseph Johnson received final payment for the bridge in June 1861.

Gilpin's Falls Covered Bridge served travelers for three-quarters of a century, until it was bypassed in 1936 and closed to traffic. A few years later, the nearby community of Salisbury expressed an interest in purchasing the covered bridge for their city park. Public sentiment led state and county officials to save the structure as a local landmark and tourist attraction. Yet, while the bridge remained intact, little money was available to maintain it.

After the bridge's roof collapsed in 1958, the Historical Society of Cecil County and Maryland's State Roads Commission rehabilitated the span at a cost of $11,000. Harry C. Eastburn & Son of Newark, Delaware, did the repairs. The bridge was rededicated on October 1, 1960. The span underwent additional repairs in 1971 following several incidents of vandalism.

In 1989, the Maryland State Highway Administration transferred ownership of the bridge back to Cecil County, along with a $50,000 grant for repairs. A 1997 engineering study revealed significant structural problems and insect infestation. The bridge was subsequently closed to pedestrian use, until it received a comprehensive rehabilitation in 2009-10.

BUILDER

Joseph George Johnson (1831-1900) was born in Cecil County, Maryland. He was the eldest son of Benjamin Johnson, a farmer, and his wife Mary. Joseph worked on the family farm in his youth. He married in 1857 and lived in Bay View with his wife, Mary (b.1835). The couple had six children: Emma (b.1857), Florence (b.1859), George (b.1861), Sallie (b.1863), Jennie

[4] *Cecil Whig*, September 8, 1860.
[5] Cecil County Historical Society, "Gilpin's Falls Covered Bridge 1860," typed manuscript, c.1992.
[6] *Cecil Whig*, December 15, 1860.

(b.1865) and Clinton (b.1869). The 1860 Federal census lists Johnson's occupation as "master carpenter."[7] The family moved to Baltimore sometime between 1870 and 1880, when Joseph began work as foreman of carpenters for the Baltimore & Cumberland Valley Railroad.[8] His occupation was listed as "carpenter" in the 1880 Federal census and "bridge builder" in the 1880 Baltimore city directory.[9] In addition to the Gilpin's Falls Covered Bridge, Johnson reportedly built the H.S. Stites Mill and several other bridges in Cecil County.[10] Joseph Johnson's son George became a builder, while his son Clinton was a civil engineer.[11]

REHABILITATION

Rehabilitation of the Gilpin's Falls Covered Bridge began in June 2009 and was completed in spring 2010. The information presented in this section is a result of numerous interviews with Timothy (Tim) Andrews, the lead timber framer on the project, a knowledgeable and skilled craftsman of wood bridge construction. These interviews were conducted during repeated visits by the HAER field team while completing extensive documentation at the site.[12]

In 2008, due to decay and recent flooding, Gilpin's Falls Covered Bridge (Figure 1) was in danger of collapse. The Cecil County Historical Society initiated a grassroots effort to save the bridge, resulting in a rehabilitation project funded by the Federal Highway Administration. Work began in summer 2009 to repair truss failures and areas of rot threatening to destroy the structure. Tim Andrews, of Barns and Bridges of New England, worked as sub-contractor for Kinsley Construction Inc. Andrews, a fourth-generation carpenter and master bridgewright, directed the repairs, working alongside timber framers William Truax and Jeremy Woodliff.

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[7] U.S. Federal Census, Cecil County, Maryland, 1860.
[8] "Baltimore and Chambersburg, Pa.," *Baltimore Sun*, August 8, 1881.
[9] U.S. Federal Census, Cecil County, Maryland, 1880; Baltimore City Directory, 1880.
[10] "Gilpin's Falls Covered Bridge 1860," typed manuscript.
[11] U.S. Federal Census, Baltimore County, Maryland, 1900 and 1910.
[12] This section was developed from discussions between Jeremy Mauro and Timothy Andrews, June 2009 to July 2010. Parts of this first appeared in: Jeremy Mauro, "Preserving a Nineteenth Century Bridge in the Twenty-first Century," *Conference Proceedings, Preserving the Historic Road, Washington, D.C.,* September 9-12, 2010.

Figure 1: View of the bridge four years before the rehabilitation project. Failures in the lower chord contributed to the noticeable negative camber. W. Earl Simmers, 2005.

Summary of Damage, Structural Issues, and Causes

The HAER field team visited the site in June 2009 and recorded the existing condition of the Gilpin's Falls Covered Bridge to create a record of its historic fabric before the structure was dismantled and to examine the extent of damage in the trusses, floor and roof system. This was accomplished through hand measuring and by using a Leica ScanStation 2 laser scanner. Hand measuring resulted in detailed drawings of the timber frame connections and their conditions while the laser scanner captured overall dimensions and recorded the deformation of the bridge along the X, Y and Z axes.

Although recent flooding caused the most catastrophic failures in the truss, it was in a severely weakened state before the flood even occurred. The most apparent areas of deterioration were the ends of the arches and where the arches met the abutments. During a previous rehabilitation effort in 1959, the abutments had been rebuilt with "bedded" timbers—12" x 12" timbers inset into the face of the abutment, positioned to bear the load of the arches and embedded in concrete. Structural investigation showed that rot and insect infiltration resulted in extremely compromised embedded timbers, and the replaced sections of arches were rotten and crushed at their ends. This area of decay revealed the causes that contributed to the failure of the bridge. The force of the arches had crushed the crumbling embedded timbers, as well as the ends of the rotting arches. The movement caused by decomposition created room for the arches to extend, flatten and deform. This had the effect of partially unloading the arches and placing additional load on the truss. The additional strain on the truss, and decades of decay, crushed and damaged the joints between truss members. Initially, before rehabilitation work began, Andrews estimated that 20 percent of the truss timbers needed to be replaced, but after disassembly it became evident that the number was far higher (Figure 2).

Figure 2: This photograph of the top of a post shows the typical condition of the truss members between joints. Field photograph, Timothy Andrews, 2009.

The crush damage combined with water and insect damage in the joints resulted in the need to replace additional timbers. Damage was most common between the abutment, or jowel, of each kingpost and the diagonal. As the joints deformed through crushing, the truss lost its camber—an indication that a covered bridge is very fatigued and needs repair. The HAER field team documented 12-$^{1}/_{2}$" of negative camber along the lower chord. As the bridge sagged, the replaced ends of the arches acted as hinges instead of buttressing the arch because they had been installed with their joints aligned rather than staggered. Shear keys, designed to keep the chord from extending lengthwise, failed under increased tension forces. The ultimate effect of this was complete failure of the lower chord. According to Andrews, once the lower chord lost the ability to hold tension, the entire dead load of the bridge hung from the arches. The arches of a Burr arch truss are not intended to carry this much weight, and they consequently deformed. The HAER field team measured 1" to 8" of upstream horizontal deflection over the length of the bridge. As the bridge lost camber and bowed upstream, the north end racked. The roof system slanted downstream at the location of the worst failure in the lower chord.

Further investigation showed that the entire upper chord, collar ties, upper lateral braces, floor beams and lower lateral braces were destroyed beyond repair. The upper chord was completely decayed at every post connection due to water penetrating the roof system. Rot affected the upper struts and lateral braces as well. Andrews also noted that the upper struts, or collar ties, installed during the 1959 repair were not the correct dimensions. The 1959 collar ties were 6-$^{7}/_{8}$" thick, which was not thick enough to fit the two 4" lateral braces easily. The incorrect dimensions of the collar ties required that the lateral braces be forced into position, bowing them and prohibiting proper tensioning. Andrews explained that correct tensioning of the upper truss, achieved by adjusting the wedges located in the mortise and tenon connections, is important for a properly functioning truss. Powderpost beetles infested the floor beams, which had also been replaced in 1959. Investigation showed that the beetles had penetrated to the middle of the floor timbers. Wood-boring insects also infested the stringers below the floor planks. The lower lateral braces were not salvageable due to the tenons being destroyed during the truss failure as well as to decay.

Stabilization, Alignment and Rehabilitation Process

Timothy J. Werner, the Senior Engineer at Wallace, Montgomery & Associates, the firm hired to oversee the structural engineering of Gilpin's Falls Covered Bridge, determined that the bridge was too fragile to attempt removal from its abutments. In order to complete the preservation work while leaving the bridge in place over North East Creek, a temporary support system was constructed. This platform consisted of steel beams resting on poured concrete abutments and spanned approximately 100'. It provided a surface above the water to stage work activities and carried the weight of the bridge when it was freed from the abutments. Since the temporary platform sat just above the water level, the engineers designed it to be jacked up higher in an emergency, like another flood.

With the temporary platform in place, Andrews, Truax and Woodliff began to stabilize and realign the bridge. The first step of this process was to cross-brace the structure diagonally through its interior using chains and jacking the lower chord at panel points (Figure 3).

Figure 3: Cross chains installed to straighten the bridge in plan. Also note the deck planks are laid diagonally, which was not original. The planks were returned to the perpendicular arrangement later in the project. Field photograph, Jeremy Mauro, 2009.

The cross-bracing lessened the lateral stresses while jack supports relieved the abutments and truss of carrying the load of the bridge. This process reduced tension from the truss and made it possible to straighten the bridge laterally, which was done by tightening the cross-chains at specific points. After removing the lateral bow, a second alignment technique known as longitudinal clamping was initiated. Four threaded rods, measuring 1" in diameter, were positioned in pairs running the entire length of both lower chords and threaded through wood blocks placed near the bottom of each post. The rods acted as temporary lower chords and were tensioned by turning nuts set against the wood blocks. This technique of longitudinal clamping both "gathered back" gaps created by failures in the bottom chord and fixed the position of each post. The next step involved stacking numerous 5" x 7" x 4' timbers into tall temporary crib towers, or falsework, that raised the height of the upper chord (Figure 4). These supports carried and controlled the upper part of the truss and helped return the bridge to positive camber. Finally, a series of eight 4" x 4"s, post-tensioned with chains attached to the temporary steel platform, acted as outriggers by buttressing the bridge posts and holding them securely in a vertical position

Figure 4: Photo of the lower cord supported by falsework. The crib towers transfer the weight of the bridge to the temporary platform. Field photograph, Jeremy Mauro, 2009.

With the bridge stabilized and aligned, the crew removed the roof system, cladding, upper cross braces, lower cross braces, flooring, stringers and floor beams. At this stage it was possible to begin in-kind replacement of such components as the posts, diagonals and sections of the upper and lower chords. To make certain that the bridge would not shift position, these components were replaced one-at-a-time in a "one out, one in" fashion using a crane (when available), a backhoe with an extended boom or by hand using a come-along (Figure 5). As new truss members were inserted, the joints were finely chiseled by hand to ensure proper fit between the vertical posts and diagonal braces. After carefully replacing the truss members, the team next addressed the rotten ends of the arches. New arch segments were shaped in place against the other components they interlocked. The joints of the replacement sections of the arches were staggered to increase strength and prevent them from acting like a hinge under load.

Figure 5: Timothy Andrews (top) and Jeremy Woodliff (below) guide a new endpost into position. A crane, not shown in the photograph, is lowering the post. Field photograph, William Truax, 2009.

When reconnecting the bridge to the abutments, Tim Andrews and his crew first set the lower chord in place and then lowered all of the jacks a small amount to engage the truss with a portion of the dead load *before engaging the arches with load*. Andrew's technique of pre-tensioning the truss, instead of loading the arch at the same time as the truss, allowed it to settle into position without stressing the arches. Andrews insists that a properly tensioned and stiff kingpost truss system is essential to keep the arches contained and aligned. Without the pre-tensioning technique, the arches may bear more of the load than is optimal and become over-stressed and deformed. In broad terms, according to Andrews, the truss shoulders most of the dead load while the arches carry most of the live load. The ratio of weight between the two systems (truss and arch) is a delicate balance that Andrews ultimately used his expertise to determine.

Within the larger process of stabilization and alignment, the timber framers practiced detailed craftwork that relied heavily on hand tooling and traditional methods, since that was often the most expedient way of finely tuning the wood truss while producing higher quality results. Modern tools were often used as well, such as a crane, a chainsaw (guided by a second person holding a "stinger" handle) to initially shape timbers, modern drills and circular saws. While the timber framers used modern tools for the rougher tasks of preparing timbers, the connections, joints and overall engineering of the truss were achieved using nineteenth-century heavy timber construction technology.

Adhering to the traditional ways of wood joinery required painstaking accuracy to ensure overall strength. The team's desire to produce the highest quality wood structure that would last for many decades, if not another century, was evident in their technique and use of materials. For example, Andrews returned an entire shipment of bridge timber deemed to be poor quality because it had been milled from trees that were young and small in diameter, causing the lumber to have too many knots. Poor drying of the logs in the shipment had also resulted in excessive checking. Another example is Andrews' decision to use 10" x 16" timbers for the kingposts rather than laminating the ends of a 10" x 10" timber as the initial plan specified. All timbers were finished using hand tools, such as adzes, slicks and chisels to remove the marks of modern tools like chain saws. This was done in an effort to match the texture and appearance of the historic wood members.

Tim Andrews selected wood species depending on their function within the structure. Truss members (posts, diagonals, chords and lateral braces) and the arches were hewn from Eastern white pine because of its combination of tensile strength, resistance to rot and relative light weight. White pine was also the original truss material. Secondary components that rested on the truss, such as sleepers and the flooring, were originally mixed hardwood species like oak. Andrews, considering that the bridge would be closed to vehicular traffic, chose to replace the flooring and sleepers with Southern yellow pine, a lighter wood that is still very durable, because he was confident that reducing the weight carried by the truss would significantly extend the bridge's service life. Southern yellow pine also has the advantage of being a local species, so it was available at a nearby mill. Bridge components with relatively low load demands and that would be more easily replaced than truss members, such as knee braces and rafters, were made from tulip poplar —a locally grown, lightweight, fast growing and cost-effective species. The bridge was clad with Atlantic white cedar siding and Western red cedar shingles due to their superior resistance to decay from weather. Where strength and rot resistance was essential and weight was not a concern, like the treenails, pegs and skewback wedges, black locust was used.

Solutions-Improvements in Design

From the outset of the rehabilitation project, it was understood that the historical significance of the bridge would depend upon making in-kind replacements of components. Historical precedent was strictly adhered to with a few exceptions. As Andrews proceeded with the rehabilitation work, he not only considered the bridge's historical significance, but also its structural integrity and longevity as well. Andrews was able to rehabilitate the structure to what may be considered a historically appropriate state while making a number of improvements to certain details of the bridge. These changes are described below both as an example of practical improvements to lengthen the lifespan of a covered bridge and as examples of decisions made during the construction process that took historic precedent into account.

First, in order to avoid the weaknesses of the 1959 abutment and arch connection, Andrews designed a connection based on historic "skewbacks." Traditionally, Burr truss bridges were constructed with arch ends resting flush on stone skewbacks, which are stone footings that protrude from the abutment and are shaped to the appropriate angle (or skew) to receive the ends of the arches. Robust skewbacks are capable of carrying the significant load of the arches with minimal deterioration over time and keep moisture to a minimum at the vulnerable ends of the arches. The bridge did not have skewbacks when it was documented in 2009, and it is not known if they originally existed. In 2009, the abutment and truss connection used timbers embedded in concrete. As was detailed earlier in this section, the bedded timbers disintegrated quickly compared to other bridge components, which caused several problems. As a solution to the problems resulting from the previous timbers being embedded in the abutment face, new concrete skewbacks were designed that allowed for air circulation around the arch and post ends to inhibit rot. To discourage water from penetrating the arches and bottoms of the posts, black locust was installed under the bottoms of the posts and at the ends of the arches.

The second improvement to the original design, and what Andrews considers to be the most important one, was the doubling of the length of the shear keys in the lower chord. The longer shear key design greatly increased the strength of the keys compared to the original design where the keys failed in all but one location during the flood. The keys were lengthened from 4'-0" to 8'-0". Other components were strengthened as well. The knee brace and collar ties were returned to their original dimensions of 4" x 5" and 8" x 7", replacing the smaller ones that were installed during the 1950s. The siding was also returned to its original stout thickness of $^5/_8$".

To reduce debris on the flooring and to encourage air circulation around the bottom chord, Andrews placed spacers between the lath frame and the lower chord. This created a gap that allowed debris to be swept from the flooring. A similar gap was introduced at the connection between the arches and floor beams so dirt could be flushed through rather than gather at the vulnerable spot between the arch and beam. The sills of the shelter panels were placed on "chairs"—small wood blocks—to keep them from contacting the cement stem walls.

The initial plan for the rehabilitation of Gilpin's Falls Covered Bridge specified using preservative-treated lumber (Chromated Copper Arsenate) for *all* replacement timber. Andrews, working with project engineer Tim Werner, modified the work plan to call for the use of preservative-treated lumber only in the 1" x 3" roof sheathing. Andrews made the case that the treated lumber would be a drastic change from the existing historic timber and that the high moisture content in treated lumber would create structural problems as the wood dried and shrank considerably. Rather than over-rely on preservatives, Andrews argues, a better maintenance plan is to regularly clear debris and replace the sheathing and roofing when needed.

Andrews did use wood preservative in targeted locations that are especially vulnerable to rot, such as the connection between the post and diagonal. Copper Naphthenate was brushed in these areas before final assembly. Additional chemicals used on the bridge were a fire retardant and an insect repellant. Nochar Fire Preventer (NFP), a water-based, non-toxic, non-skin irritant, biodegradable flame retardant, was brushed onto all wood surfaces from the top chord down. To repel insects from the bridge and prevent them from causing widespread damage again, the insecticide Boracare (Nisus Corporation) was applied to the weather panels on both ends of the bridge. Boracare is a wood treatment with low toxicity and low environmental impact because its active ingredient is borate mineral salt rather than more harmful chemicals. The insecticide had to be brushed on before the fire retardant to create the most effective barrier.[13]

Improvements were also made in the fasteners located on the lower chord. Galvanized bolts of the same dimensions replaced the nearly disintegrated iron ones. In an effort to reconstruct some of the original hardware of the bridge, several of the damaged iron bolts were brought to a local blacksmith, who wrought them into 8" iron spikes that were used to fasten the lower ends of the knee braces to the posts. The spikes replaced the large wire cut nails that were not original to the bridge, and matched in detail those spikes used by the original builder to attach the top horizontal nailer to the truss.

Replacing Historic Fabric

When making the decision to keep or replace historic components of the bridge, several factors must be weighed. The contractor must put forth methods of treatment that balance the imperative to preserve as much historic fabric as possible with the need to ensure the structural soundness of the truss for safety and longevity.

Andrews approached the decision to replace or to keep historic fabric by implementing a series of evaluations in specific order. The most important evaluation when rehabilitating a historic wooden bridge, Andrews asserts, is structural integrity. The first step when considering a bridge component is to evaluate if it can function structurally. If it cannot, the second question should be—is there a repair remedy? If no repair remedies are appropriate, replace the member. An example of a preservation treatment is the Dutchman repair, or clothespin scarf, which was used to repair a compromised diagonal brace (figures 6 and 7). Confirming the structural integrity of every member of the truss is also important for longevity. With the exception of wear items such as the siding, roofing and floor planks, Andrews expects the truss to serve well past fifty years with no need for intrusive maintenance.

[13] Nochar Corporation, "Nochar's Fire Preventer," http://nochar.com/nfp/, accessed November 2012; Nisus Corporation, "Bora-Care," http://nisuscorp.com/builders/products/bora-care, accessed November 2012.

Figure 6: Historic fabric was repaired rather than replaced whenever possible. The photograph shows a repair to the end of a diagonal brace. Field photograph, Christopher Marston, 2009.

Figure 7: A "Dutchman" repair to the arch is shown. This repair method preserved as much historic fabric as possible while ensuring structural reliability. Field photograph, Christopher Marston, 2009.

The rehabilitation work completed at Gilpin's Falls Covered Bridge provides an example of best practices for covered bridge rehabilitation. Andrews and his crew implemented methods of treatment that addressed and solved widespread structural failures, yet did not significantly harm the historic integrity of the bridge. The rehabilitation team examined all options during each step of the process and found solutions to best ensure longevity, preserve as much historic fabric as possible and maintain the bridge's historical engineering (spanning 99' with a wood truss). Another considerable factor that contributed to the success of the rehabilitation was the expert woodcraft carried out by Andrews, Truax and Woodliff. Their knowledge of traditional methods of timber framing created continuity with the nineteenth-century construction methods of the bridge. While the materials of the bridge were replaced "in-kind," one may also consider many of the wood crafting techniques performed to be "in-kind" with its original construction date as well. As a result of this work, Gilpin's Falls Covered Bridge is prepared to enter its next century of service with its historic significance intact and its historic engineering preserved.

ENGINEERING REPORT

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1. INTRODUCTION

Gilpin's Falls Covered Bridge is the longest remaining covered bridge in the state of Maryland and was listed in the National Register of Historic Places in 2008. It is also a classic example of the Burr truss form, the first in a series of successful truss patents issued in the early part of the nineteenth century. Other successful patents included the Town lattice truss (1820), the Long truss (1830) and the Howe truss (1840). Being the first, the Burr truss differed from later patents in its strong link to the craftsman tradition, a tradition characterized by the use of large timbers and traditional wooden joinery. Later patents, particularly those of Town and Howe, successfully increased the efficient use of both materials and labor.[15]

A Burr truss consists of a multiple kingpost truss with a two-hinged, superimposed arch springing between abutment faces and connected to the truss at each post with a through-bolt. The superposition of these two systems creates a highly indeterminate structure – one whose form could not have been analyzed by bridge builders of the day. In recent years, engineers have used modern structural analysis tools to study the behavior of the Burr truss, but their conclusions have varied. Some say that the truss carried most of the load, but the arch was necessary to provide stiffness for long spans and long-term deflections, while others say that the arch carried a majority of the load but required the stiffening truss to prevent excessive instantaneous deflections under live loads.

In 2009, the bridge was in danger of collapse, but rehabilitation commenced thanks to a grant from the Federal Highway Administration, as has already been described. Whereas past experimental work on Burr trusses measured the bridges' response to asymmetric live loads, rehabilitation provided engineers with the opportunity to measure the effects of dead load on the truss and arch. These effects changed as rehabilitation progressed, highlighting the importance of construction technique on the performance of the bridge. This engineering report reviews the experimental and numerical work undertaken to measure and model the bridge as it was rehabilitated.

2. THEODORE BURR AND THE DEVELOPMENT OF THE BURR TRUSS

Theodore Burr was born in Torringford, Connecticut, in 1771 and moved to Oxford, New York, in 1792. After starting out building dams and mills in the vicinity of his home in Oxford, Burr began successfully building several long-span wooden bridges in the early nineteenth century. Among these was the 175' "Union Bridge" built in 1804 across the Hudson River between Waterford and Lansingburgh, New York (Figure 1). The design consisted of a parallel chord truss with braces and counterbraces in each panel and a superimposed arch springing between the abutments. The bridge carried traffic until 1909, when it was destroyed by fire. Around the same time that he was working on the Union Bridge, Burr also built a bridge over the Delaware River near Trenton, New Jersey. This bridge differed significantly from the Union Bridge, as it had tied wooden arches supporting the deck via iron chains (Figure 2). An iron bridge replaced the Trenton Bridge in 1876. In the Mohawk River Bridge, built in 1808, Burr experimented with a wooden suspension form (Figure 3). While the bridge stood until 1873, significant deflections

[15] Ithiel Town, U.S. Patent No. 3169X, issued January 28, 1820; Stephen Harriman Long, U.S. Patent No. 5862X, March 6, 1830; William Howe, U.S. Patent No. 1711, issued August 3, 1840.

became a problem after only twenty years, and additional piers were needed to support the bridge at the middle of each span.[16]

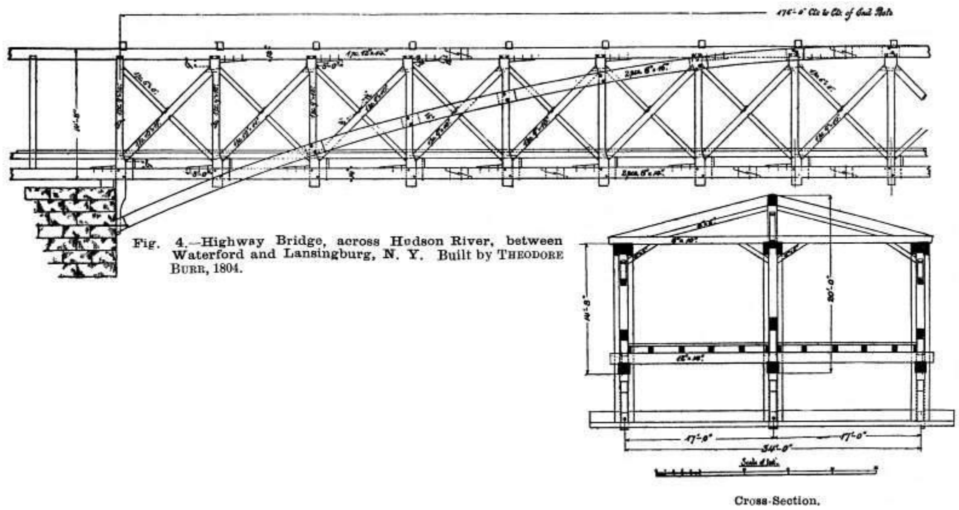

Figure 1: Burr's Union Bridge across the Hudson River between Waterford and Lansingburgh, New York (1804). From Theodore Cooper, "American Railroad Bridges," *Transactions of the American Society of Civil Engineers* 21 (July 1889): Plate VI.

After some experimentation with form in the first decade of the nineteenth century, Burr seemed to settle on a form similar to his Union Bridge in Figure 1. His arch-truss system was employed in five bridge crossings on the Susquehanna River: Northumberland-to-Sunbury (1814), Harrisburg (1816), McCall's Ferry (1815), Columbia-Wrightsville Bridge (1815) and Berwick Bridge (1815). Unfortunately, only the Northumberland and Harrisburg Bridges withstood the river's power (Figure 4); McCall's Ferry Bridge was taken down by ice just three years after it was built, Columbia-Wrightsville Bridge was destroyed in a flood in 1832, and the Berwick Bridge met a similar fate in the winter of 1836. The collapse of McCall's Ferry Bridge was particularly devastating to Burr, for its span length of 360' exceeded by 20' Lewis Wernwag's "Colossus of Fairmount" completed three years earlier.[17]

[16] Jeanie Petersen. "Oxford celebrates Burr House Bicentennial and Covered Bridge Resource Center grand opening," *The Evening Sun*, July 19, 2011, http://www.evesun.com/news/stories/2011-07-19/12663/Oxford-celebrates-Burr-House-Bicentennial-and-Covered-Bridge-Resource-Center-grand-opening-/, accessed October 16, 2012.

[17] The design of McCall's Ferry was likely a variation of a trussed arch, not the classic Burr truss form. The bridge is described in a letter to Reuben Field (February 26, 1815), published in "McCall's Ferry Bridge," *Niles Weekly Register*, November 18, 1815, 200-201. See also, Hubertis M. Cummings, "Theodore Burr and His Bridges across the Susquehanna," *Pennsylvania History* 23, no. 4 (October 1956): 476-486.

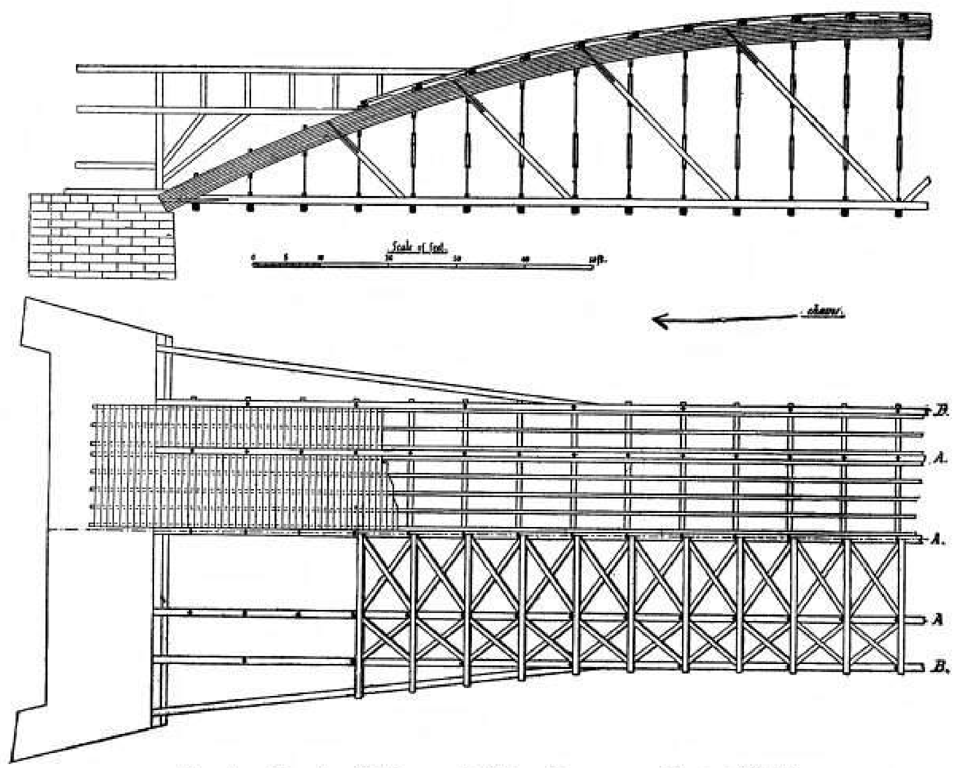

Figure 2: Burr's bridge across the Delaware River south of Trenton, New Jersey (1806). From Theodore Cooper, "American Railroad Bridges," *Transactions of the American Society of Civil Engineers* 21 (July 1889): Plate VII.

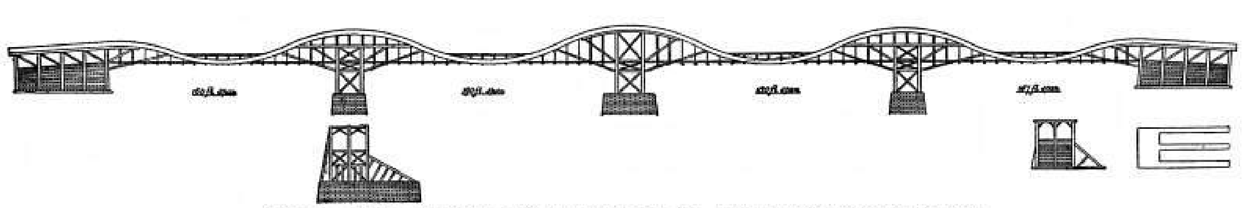

Fig. 7.—Mohawk Bridge at Schenectady, N. Y. Built by THEODORE BURR, 1808.

Figure 3: Burr's bridge across the Mohawk River at Schenectady, New York (1808). Theodore Cooper, "American Railroad Bridges," *Transactions of the American Society of Civil Engineers* 21 (July 1889): Plate VIII.

Burr took out his first patent for a wooden truss arrangement in 1806, but the records were destroyed when the U.S. Patent Office burned in 1836. Fortunately, among the 2,845 patents that were restored after the fire was Burr's second patent, awarded in 1817. Burr's 1817 patent, shown in

Figure 5, closely resembles his 1804 Union Bridge (Figure 1), the longest-surviving of his early long-span bridges, as well as several of his bridges across the Susquehanna River, including the Harrisburg ("Camelback") Bridge (Figure 4). The Burr truss, as it came to be known, combines a multiple kingpost truss and a two-hinged parabolic arch, springing from the abutments. The truss is positioned between two arch halves, which reduces eccentric loading of the arch. While the patent shows only a single arch, Burr's Harrisburg Bridge (Figure 4) employed multiple arches, suggesting Burr's intention for the design to be modified with the addition of multiple arches for longer spans.

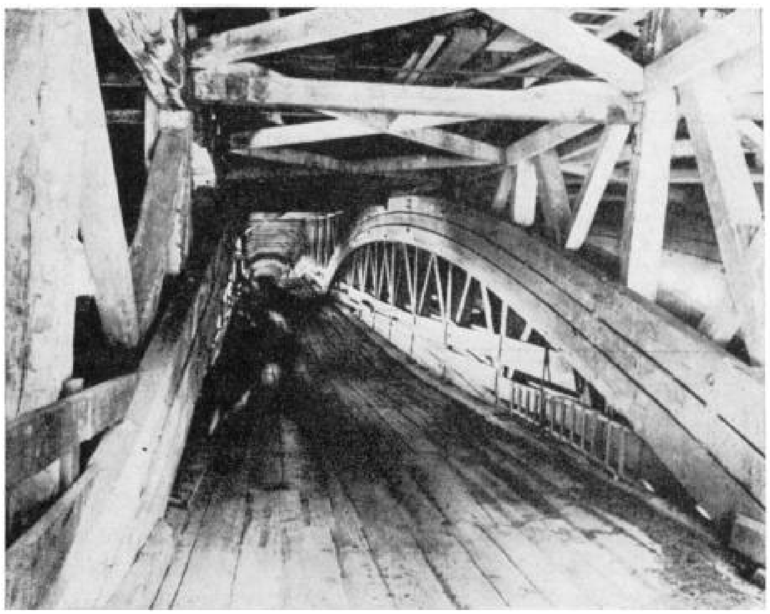

Figure 4: Interior of Burr's "Camelback Bridge" (Harrisburg Bridge) across the Susquehanna River at Harrisburg, Pennsylvania (1816). Courtesy of The Theodore Burr Covered Bridge Society of Pennsylvania.

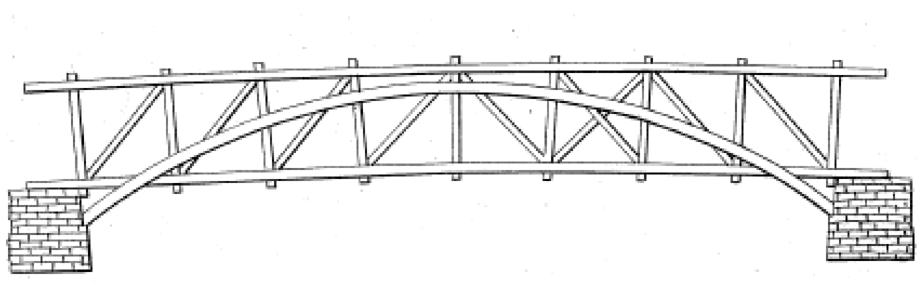

Figure 5: Patent drawing that accompanied Burr's original 1817 patent. U.S. Patent No. 2769X, issued April 3, 1817.

2.1 Construction of a Burr Truss

Theodore Burr discussed his method for constructing the Burr truss in his patent application. After describing the erection of the truss and lateral bracing, Burr made some comments on the arch:

> …the arches are the last principal timbers that are to be raised; they may be notched a little where they cross the chords and where they cross the king posts and braces, if thought best, but seldom necessary on the posts or braces. The arches may take their rise from below the chord, or at the chord line, as may be required to give the direct curve and to rise to the top of the crown plate or towards it, and even above it in very long spans if desired and may be double or single, if double one arch on each side of the king post and braces; they are put on so as to leave the king post between the arches when double, if they are left single to be put on that side of the king post that suits best…[18]

While the description provides some details of Burr's design intentions, questions remain regarding the sequencing of construction, specifically whether or not the truss was supported at the time the arches were "raised." The answer to that question would have affected how the bridge carried load, particularly long-term.

In his 1851 book, *General Theory of Bridge Construction*, Herman Haupt describes the Pennsylvania Railroad Viaduct across the Susquehanna River, located 5 miles north of Harrisburg, Pennsylvania. The bridge superstructure was comprised of 160'-0" span Howe trusses with superimposed arches; thus the challenges in identifying the distribution of load between the arch and truss were the same as those encountered in the Burr truss, and the question over construction sequencing was just as relevant. Haupt describes how construction would define the load distribution between the two systems.

> If, for example, a truss be constructed, and the false works removed before the introduction of the arches, if the latter be bolted to the posts, the weight of the whole structure is sustained by the truss itself, and the arches will not bear a single pound, unless they are called into action by an increased degree of settling in the truss.
>
> …
>
> Again; if we suppose the arches be connected with the truss before the removal of the false works, and the joints be equally perfect in

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[18] Llewellyn Nathaniel Edwards, *A Record of History and Evolution of Early American Bridges* (Orono, Maine: C. H. Edwards, 1959), 51-52.

> both systems, there is a prospect of a more nearly uniform distribution of the load; but even in this case, we cannot tell what portion is sustained by each system, because this will depend upon their relative rigidity.[19]

In Indiana, around the same time that Haupt's book was published, two families of bridge builders apparently adopted these differing approaches. The Kennedys were a three-generation family of bridge builders. Archibald M. Kennedy, sons Emmett and Charles, and grandsons Karl and Charles R. together built about sixty covered bridges in Indiana and Ohio between 1870 and 1918, including Kennedy Bridge in Rushville, Indiana, and Forsythe Bridge near Moscow, Indiana.[20] The Kennedy family's construction technique is documented in *Indiana Covered Bridges thru the Years.*

> At the bridge site falsework was erected at various points across the stream to support wooden blocks which in turn supported the lower chord. The framing of the truss proceeded with the erection of the vertical and diagonal timbers to connect the upper and lower chords. A derrick lifted the heavy timbers into place. Upon completion of the two trusses the structure was raised slightly so that the blocks could be removed. Then the span was carefully lowered until it was supporting its own weight. Bridges were built with a slight elevation in mid-span, called camber, so that when supports were removed, the flooring was level throughout the structure. Minor adjustments of the span were possible by driving in, or loosening, the small wedges in the top lateral and floor support bracings. The falsework was in place until the workmen had finished the siding and roof.[21]

Like the Kennedy family, Joseph J. Daniels was an Indiana bridge builder who built about sixty covered bridges over his career, stretching from 1855 to 1900. Almost all of the bridges were of the Burr truss form, including the Jackson Covered Bridge in Parke County, Indiana, the longest single-span covered bridge still carrying vehicular traffic.[22] While Daniels was a contemporary of Emmett and Charles Kennedy, who built Burr truss bridges in the same region of Indiana, his approach to construction differed in that he "erected the arches alongside the truss timbers, but did not fasten them until the false work was removed and the main framework had settled into place." The following excerpt from the "Specifications for a Wooden Truss Bridge," which likely originated from Daniels himself, describes this construction detail: "When the bridge is raised as far as described above, the blocking shall be knocked out so that it shall be self-

[19] Herman Haupt, *General Theory of Bridge Construction* (New York: D. Appleton and Co., 1870), 174-175.
[20] Historic American Buildings Survey (HABS), National Park Service, U.S. Department of the Interior, "Kennedy Bridge," HABS No. IN-24-1, and Historic American Engineering Record (HAER), National Park Service, U.S. Department of the Interior, "Forsythe Bridge," HAER No. IN-106.
[21] George E. Gould, *Indiana Covered Bridges through the Years* (Indianapolis: Indiana Covered Bridge Society, Inc., 1977), 11-16.
[22] HAER, National Park Service, U.S. Department of the Interior, "Jackson Bridge," HAER No. IN-48.

sustaining. Then the arches shall be brought on out to proper lengths and adjusted to their places. When they shall be bolted to each post by 7/8" bolts."[23]

Traditional wooden joints take time and load to settle into place. As they settle, the structure gains stiffness while losing some of its camber. This deliberate pre-stiffening of the truss under its own weight suggests that Daniels understood the importance of the truss in stiffening the arch. Daniels' approach was likely more noticeable in the long-term behavior of the bridge. A Kennedy bridge, being less stiff, would have led to an early settlement of the truss. The arch would have to resist this settlement by absorbing a portion of the dead load that the truss had been carrying. Thus, in a Daniels bridge, the arch would be less-stressed over time, as the truss would play a more significant role in carrying dead load than in a Kennedy bridge. For the rehabilitation of Gilpin's Falls Covered Bridge, Tim Andrews followed Daniels' approach, allowing the truss time to settle into place under its own weight before connecting the arches.

Haupt concludes that no matter which construction method is employed, the end result is the same:

> it will generally happen that after a bridge has been a long time in operation, the two systems bear very unequal portions, and when the truss itself is not so constructed as to be susceptible of adjustment, the arch almost always sustains the whole weight of the bridge, and its load.

3. REVIEW OF PAST ENGINEERING STUDIES ON BURR TRUSS BRIDGES

Engineers who have studied Burr trusses have come to seemingly different conclusions after evaluating the arch-truss interaction in the Burr truss. In the study of Barrackville Bridge (HAER No. WV-8) in Barrackville, West Virginia, Emory Kemp and John Hall concluded that the multiple kingpost truss was capable of supporting its own weight, but required the arch for longer spans (>70') to stiffen it against large deflections.[24] Conversely, engineers who studied Pine Grove Bridge (HAER No. PA-586) near Oxford, Pennsylvania, proposed that the arch carries a majority of the dead load but requires the truss to stiffen the arch against significant deflections under concentrated live load forces.[25] In a follow-up study on Pine Grove Bridge, engineers found further evidence to suggest that the arch is the dominant load-carrying system.[26] Their findings are reviewed in more detail in the following sections.

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[23] "Specifications for a Wooden Truss Bridge," J.J. Daniels papers, Indiana Historical Society, courtesy of Joseph Conwill.

[24] HAER, National Park Service, U.S. Department of the Interior, "Barrackville Bridge," HAER No. WV-8; Emory L. Kemp and J. Hall, "Case Study of Burr Truss Covered Bridge," *Engineering Issues: Journal of Professional Activities* 101, no. 3 (1975): 391-412.

[25] HAER, National Park Service, U.S. Department of the Interior, "Pine Grove Bridge," HAER No. PA-586; Dylan Lamar and Benjamin Schafer, "Structural Analyses of Two Historic Covered Wooden Bridges," *Journal of Bridge Engineering* 9, no. 6 (2004): 623-633.

[26] Rachel H. Sangree and Benjamin W. Schafer, "Field Experiments and Numerical Models for the Condition Assessment of Historic Timber Bridges: Case Study," *Journal of Bridge Engineering* 13, no. 6 (2008): 595-601.

3.1 Barrackville Bridge

Lemuel and Eli Chenoweth built the Barrackville Bridge, a 130'-0" Burr truss, in 1853 (Figure 6). In 1975, Emory Kemp and John Hall studied the bridge by building and analyzing a frame model in ICES STRUDL II, an early structural analysis program. The authors analyzed their model of the bridge truss, both with and without the arch and under dead and live loads to better understand how the presence of the arch affected the bridge's behavior. They found that without the arch, the truss was capable of supporting the loads by itself. The addition of the arch tremendously benefits the truss members with a 60 percent decrease in the upper chord compressive force and a 90 percent decrease in the lower chord tensile force; further, their analysis found that the arch carried about 60 percent of the compressive force of the upper chord.

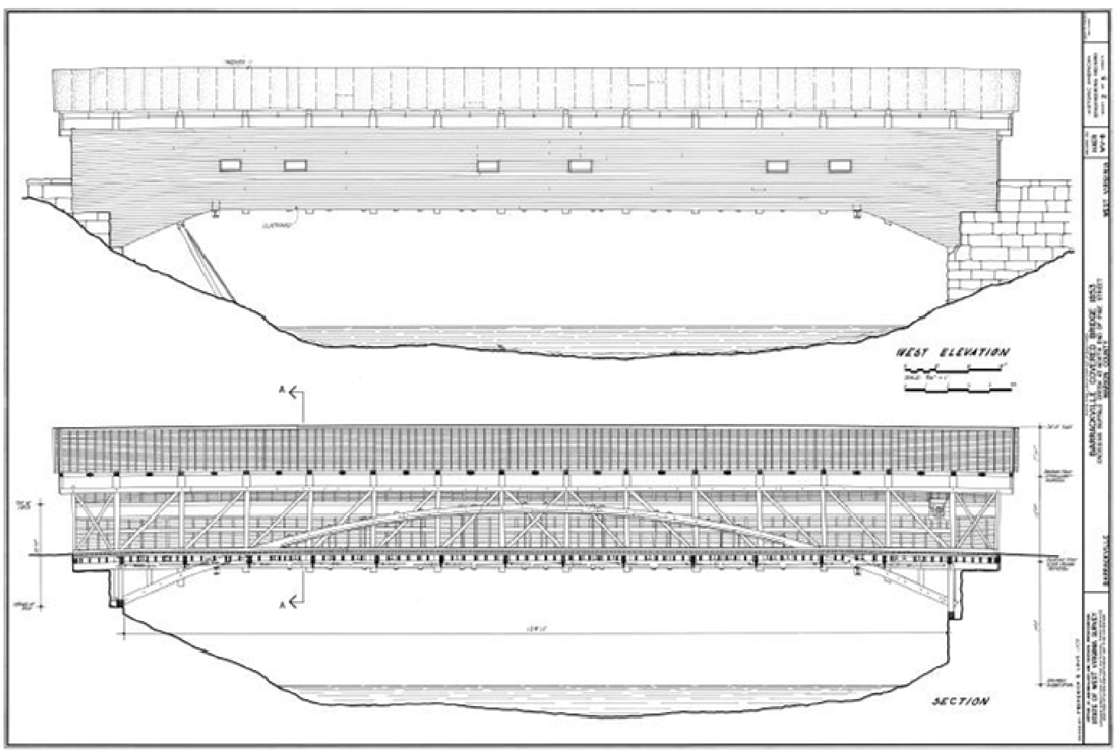

Figure 6: Frederick R. Love, delineator, Barrackville Bridge, HAER No. WV-8, Sheet 2 of 3, 1973.

In addition to the arch's impact on member forces, midspan deflections were significantly reduced with the addition of the arch to the model. This led the authors to speculate that since the truss members were strong enough to support the loads without the arch's assistance, the arch was added to stiffen the truss. This argument is emphasized by further discussion related to the long-term effects of creep and joint integrity on the stiffness of the truss, which would require assistance of the arch in the future to provide a means for stiffening the system. Kemp and Hall reference the traditional builder's rule of thumb "for spans over 70'-0", an arch should be added," which suggests that traditional builders felt that a longer span length required the addition of an arch for reinforcement against excessive deflections and forces.

3.2 Pine Grove Bridge

Engineering researchers studied Pine Grove Bridge as part of the first phase of HAER's National Covered Bridges Recording Project in 2002. Built by Elias McMellen in 1884, Pine Grove Bridge is a two-span Burr truss spanning the Octoraro Creek near Oxford, Pennsylvania (Figure 7). Similar to Kemp and Hall's analysis, Dylan Lamar and Benjamin Schafer separated the arch-truss system into its two parts as a means of better understanding the whole. Unlike their predecessors, however, they also modeled the arch by itself, demonstrating that it, too, is inherently flexible without the presence of the truss to stiffen it.

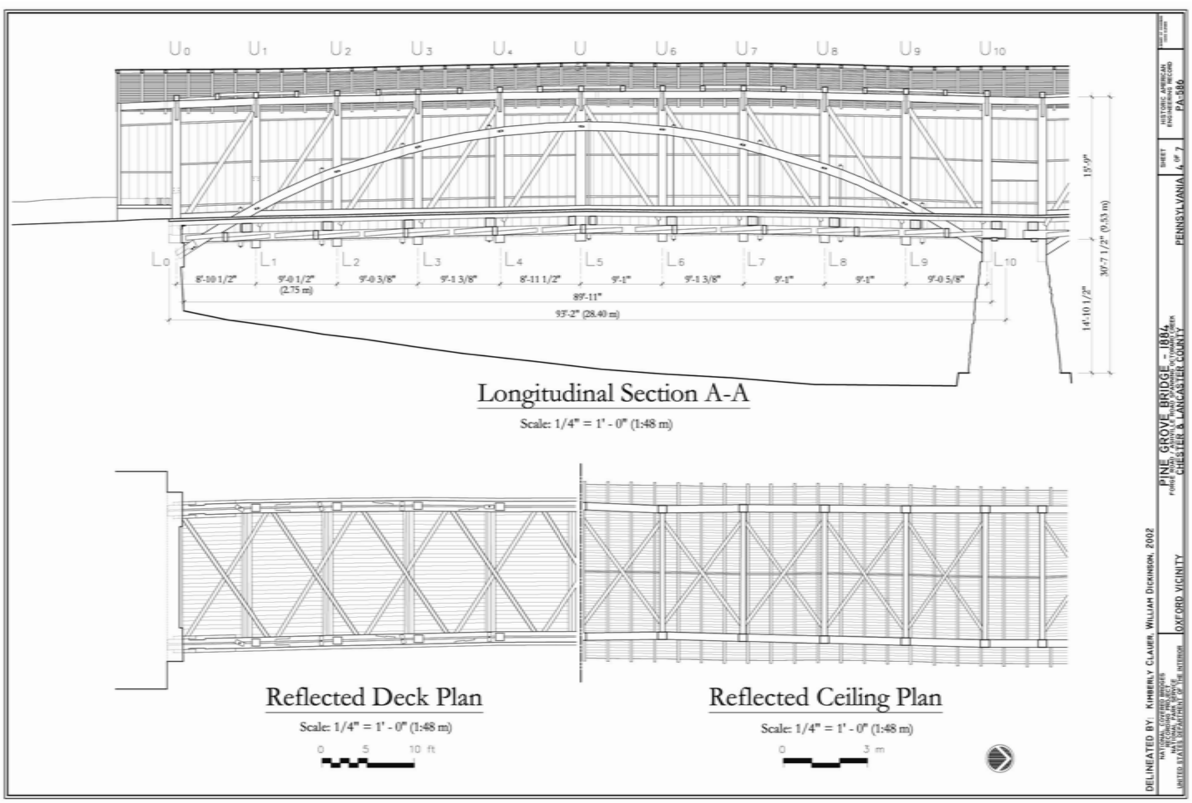

Figure 7: Kimberly Clauer and William Dickinson, delineators, Pine Grove Bridge, HAER No. PA-586, Sheet 4 of 7.

Modeling the system as a whole resulted in a 33 percent decrease in arch stress and a 77 percent decrease in truss chord stress, with the end result being a maximum arch force 350 percent greater than the maximum truss force. This led the authors of the Pine Grove study to conclude that under dead load, the arch was the structurally dominant component. On the other hand, under concentrated live loads applied in the middle half of the span, force is more readily carried by the truss diagonal members to transfer shear to the chords while the arch is relatively horizontal near the middle of the span and thus cannot support vertical shear forces. The truss counteracts the bending moments through the top and bottom chords and the shear forces through the diagonal members. Since dead loads in the bridge are about ten times the live loads, the arch was deemed the dominant system, but it necessitates a truss to stiffen it against asymmetric, concentrated live loading.

In 2003, a follow-up study of Pine Grove Bridge was performed using load tests to evaluate the arch-truss interaction under asymmetric, concentrated live loads. The authors hypothesized that if the truss stiffens the arch (rather than the other way around), then under a concentrated live load, deflections should resemble arch-only deflections under a similarly placed live load, but should be smaller due to the stiffening power of the truss. Conversely, if the arch stiffens the truss, then under a concentrated live load force, deflections should resemble typical truss deflections under a concentrated force, again only smaller.

Load tests demonstrated the former to be true; deflections were clearly representative of arch deflections under live load but were significantly smaller. An example of influence line data generated from the live load tests is shown in Figure 8. Here, the bridge deflection was measured at the quarter-point of the span as a truck was moved from locations at the quarter-point, midspan and three-quarter point. With the truck at the quarter-point, the deflection of the bridge was downward, as one would expect; however, with the truck at the midspan and the three-quarter points, the bridge moved upward at the quarter point. This upward deflection is evidence that the arch, not the truss, carries a more significant portion of the load, but the deflection magnitudes would be much greater (in fact the arch would be useless!) if the truss did not provide stiffness to the system.

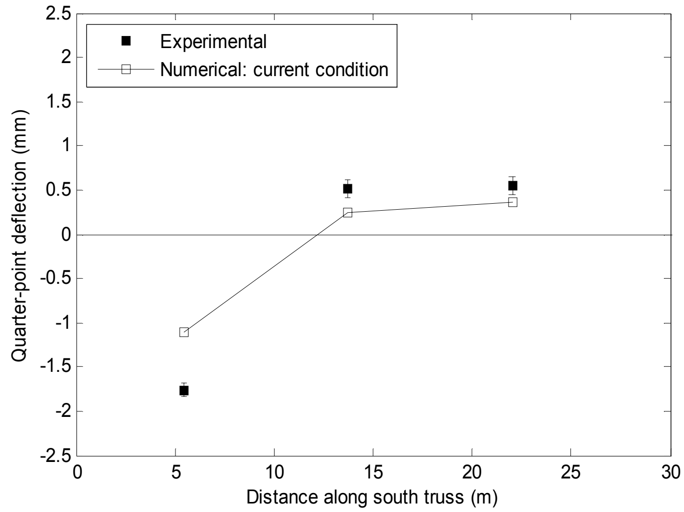

Figure 8: Experimental and numerical influence lines for deflection at the quarter-point of Pine Grove Bridge. Rachel Sangree and Benjamin Schafer, "Field Experiments and Numerical Models for the Condition Assessment of Historic Timber Bridges: Case Study," *Journal of Bridge Engineering* 13, no. 6 (2008).

4. STRUCTURAL BEHAVIOR OF THE BURR TRUSS

Structural behavior of the Burr truss is often illustrated by separating the form into its two systems, arch and truss, and demonstrating how each system behaves under a symmetric, uniform dead load and an asymmetric, concentrated live load. What these analyses demonstrate is that both the arch and the truss function efficiently under dead load, transferring load through axial tension and compression forces in the members until it reaches the abutments. Analyses also show that neither the arch nor the multiple kingpost truss is stiff enough to effectively support an asymmetric live load. The similar strengths and weaknesses of these two structural systems makes it difficult to identify which system, if either, supports a majority of the dead load, and how both systems work together to support the live load that neither could on its own.

While the uniform, symmetric dead load may be supported by either system, it is likely that the distribution of dead load between the arch and truss changes over the life of the structure. As introduced in Section 2.1 and discussed further in Section 9.4, construction sequence is probably the most influential parameter controlling the initial distribution of dead load between the arch and truss. However, even when construction methods result in a fairly uniform initial distribution (that is, the arch and the truss share the load equally), over time the viscoelastic nature of wood likely causes the truss to settle, and in doing so transfers much of its share of the load to the arch. Herman Haupt introduced this characterization of behavior in the mid-nineteenth century, and Kemp and Hall restated it in 1975.

How the two systems share the burden of live load may best be framed in a discussion of the history of braces and counterbraces in bridge trusses. In any parallel chord truss, the top and bottom chords resist the bending moments while the diagonal members resist the shear forces. A diagonal member transfers shear through compression or tension depending on its orientation within its panel, the panel's position within the truss, and the type of loading on the bridge. Under its own weight (a uniform, symmetric load, Figure 9a), which is the most significant load a wooden truss bridge bears, the diagonal members in Figure 9c resist shear force (Figure 9b) through compression and are called braces; the diagonal members in Figure 9d resist shear through tension and are called counterbraces.

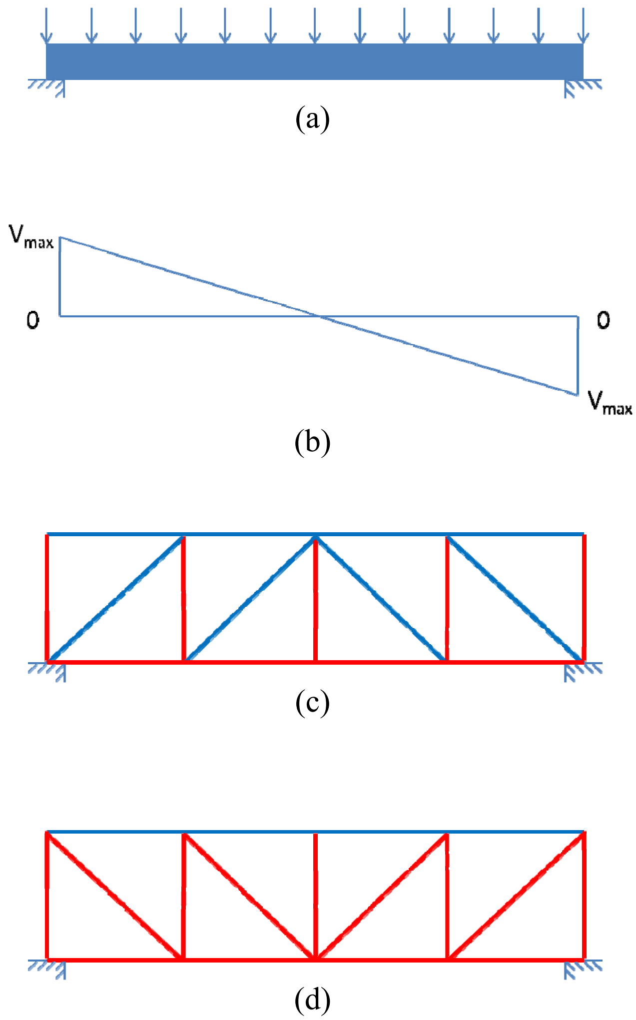

Figure 9: The effect of a uniformly distributed load on axial forces in a truss. (a) shows a simply supported beam with a uniformly distributed load; (b) shows a shear force diagram for the beam in (a); in (c), diagonal braces are oriented to resist shear forces through compression; and in (d), diagonal counterbraces are oriented to resist shear forces in tension. Note: for (c) and (d) tension members are shown in red and compression members in blue.

If early bridge builders had been able to work with steel, the choice of brace or counterbrace would have been less important because creating an effective compression or tension connection at the end of a steel member is relatively simple. However, early bridge builders worked with timber, and transferring force through a tension connection at the end of a timber member was not feasible. The result was that the most primitive truss forms relied solely on braces (diagonal members in compression under uniform dead load) and did not employ counterbraces. Examples include the kingpost, the queenpost and the multiple kingpost trusses. Up to a certain span length, the use of braces alone was adequate, but as wider crossings were made with longer bridges, the effect of asymmetric concentrated live loads necessitated the use of counterbraces to stiffen the truss when shear forces in the panels reversed direction (see Figure 10c).

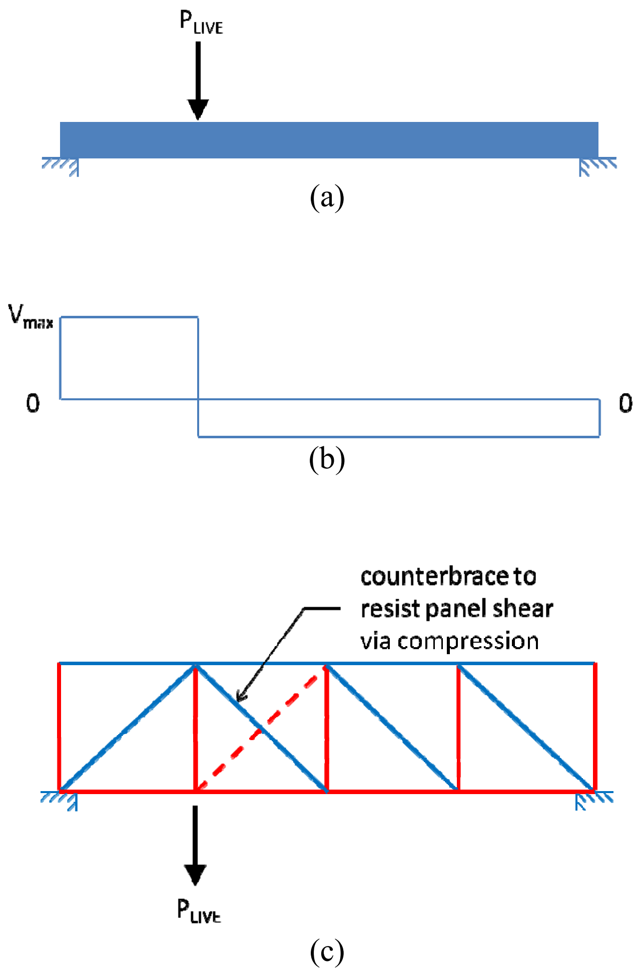

Figure 10: The effect of a concentrated, asymmetric live load on axial forces in a truss. (a) shows a simply supported beam with a concentrated live load placed at the quarter point of the span; (b) shows the shear force diagram for the beam in (a); in (c), the diagonal brace adjacent to the asymmetric live load (shown with a dashed line) experiences a reversal in stress, leaving it unable to transfer force to the lower chord. A counterbrace in this panel would provide stiffness by resisting the shear force through compression. Note: for (c) tension members are shown in red and compression members in blue.

Dead load in a wooden truss is significantly greater than live load, but the shear forces in the diagonal members that result from dead load is nearly zero at the midspan of a bridge (Figure 9b), while those that result from live load are nonzero (Figure 10b). Thus, if the live load is near midspan of the bridge, a panel that relies on braces alone could be left without any means of resisting the shear. In the worst case, this situation could cause a brace to disengage from the rest of the truss or even fall out. In the best case, the bridge would deflect significantly under live load.

Early builders of long-span wooden truss bridges, like Timothy Palmer and Lewis Wernwag, understood that their bridges would be too flexible under concentrated live load forces if they used braces alone in their truss panels. Both builders incorporated counterbraces in each panel so that panel shear could be resisted, even when the braces were relieved of their compressive force due to a reversal in panel shear. However, to keep the counterbraces from disengaging from the truss as it settled over time, Palmer and Wernwag had to incorporate other elements in their designs like arches and steel tie rods. Burr even showed counterbraces in his patent drawing and used them in his Union Bridge, but it may have become clear over time that they were not entirely necessary since *the arch, once in compression under dead load, provided the same counterbracing effect*.

A simple analysis is shown here to reveal how this works. In Figure 11a, the dead load is evenly distributed to the three lower panel points between supports, causing compression in the arch, top chord and braces, and tension in the lower chord and posts. When a live load is added to one of the panel points in Figure11b, the brace connected to that panel point is relieved of its compressive force; to be conservative we may assume that it becomes a tension member, and therefore is no longer able to resist panel shear. However, the arch, which is still compressed by the uniform dead load, assumes the responsibility of the brace and resists the shear in the affected panel caused by the concentrated live load.

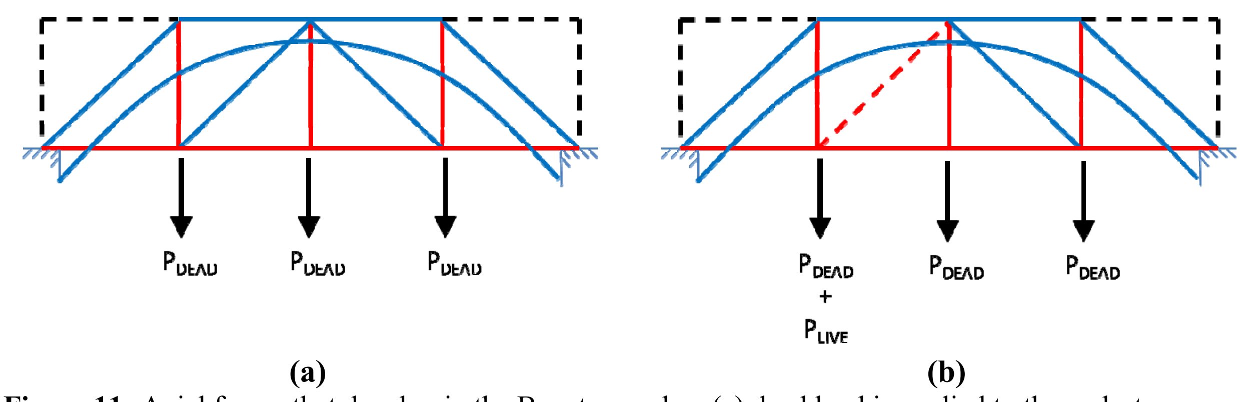

Figure 11: Axial forces that develop in the Burr truss when (a) dead load is applied to the arch- truss system before (b) live load is applied to the system. [Note: red = compression; blue = tension; black dash = zero force]

While either system is capable of resisting the dead load effects, it is critical to the resistance of live load effects that the arch carries at least some of the dead load. If the arch is not compressed prior to the applying the live load (see Figure 12a), it will not assume the responsibility of the brace. In fact, it too will become a tension member once live load is applied, as demonstrated in Figure 12b.

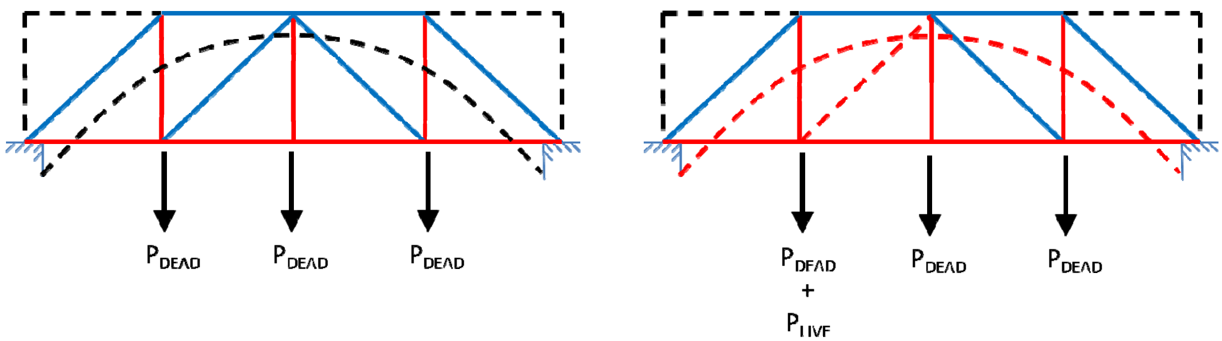

Figure 12: Axial forces that develop in the Burr truss when (a) dead load is applied to the truss before (b) live load is applied to the system. [Note: red = tension; blue = compression; black dash = zero force; red dash = tension member that cannot transfer tension, effectively a zero force member]

5. GILPIN'S FALLS BRIDGE

5.1 Original Configuration

The original advertisement for Gilpin's Falls Covered Bridge was printed in an 1860 edition of *The Cecil Whig* and called for: "The wood work to be on the "Burr" plan; span 100 feet; width from out to out 17 feet; 14 feet from string pieces to square; to have double ribbed segments, double arch, and double string pieces."[27]

Joseph G. Johnson won the contract, and he built a bridge to the specifications provided above. The multiple kingpost truss was constructed in ten 9'-10"-wide panels, producing a span length of 98'-4" between end posts. Counterbraces were not provided in any of the panels. The arches sprang from the abutment faces, and were originally connected to the truss at the posts via $^{1}/_{2}$" – deep notches cut into the posts and $^{5}/_{8}$"-diameter wrought-iron threaded rods. Arches were constructed in segments (a.k.a. leaves) that spanned two panels. The leaves were staggered to prevent any weak points along the span. Member dimensions are provided in Table 1 and material properties for Eastern white pine, the timber used in Gilpin's Falls Bridge, are in Table 2.

The community continued to use the bridge until 1936, when State Route 272 bypassed it to the south with a reinforced concrete bridge. No longer used for vehicular traffic, the bridge was neglected and fell into disrepair. At one time, there were plans to move the bridge to a city park in Salisbury, Maryland. At that time, covered bridge historian Richard Sanders Allen wrote:

[27] "To Bridge Builders and Contractors," *The Cecil Whig*, August 18, 1860.

> Little by little Gilpin's Rocks Bridge deteriorated: a shingle loose, a board off; then the quiet drip, drip, drip of rain to turn the exposed wood to punk. In the heavy snows of early 1958 the roof collapsed. A year later the bridge had outlasted its concrete successor, but only as a skeletonized ruin. It's probably far too late to take it to Salisbury now. [28]

As predicted by Allen, Salisbury never acquired the bridge; however, in 1958 the Historical Society of Cecil County and the State Roads Commission of Maryland raised funds for a major rehabilitation. Unfortunately, by the 1980s the bridge was once again in poor condition. Though the community fought to keep it, they could only raise enough money for minor repairs. In 2007, Gilpin's Falls Covered Bridge received funding from the Federal Highway Administration, which enabled its rehabilitation.

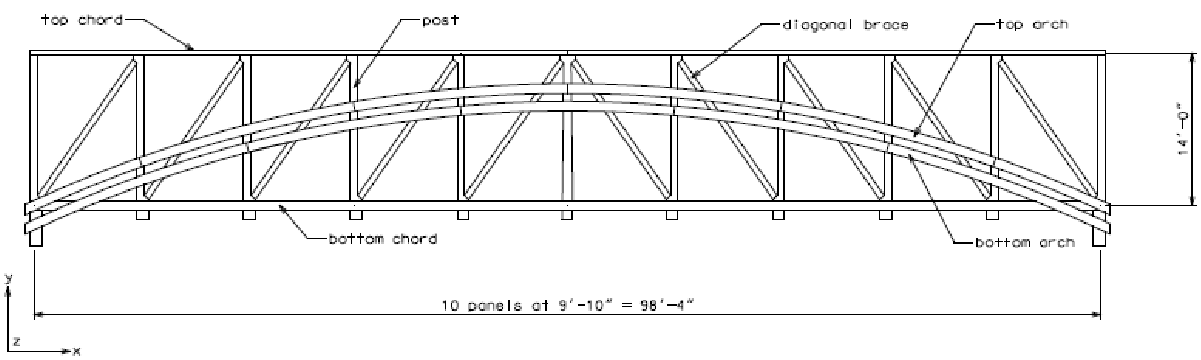

Figure 13: Two-dimensional drawing of Gilpin's Falls Covered Bridge (note: camber is not drawn, but it would have been present in the original configuration).

Table 1: Member dimensions

Member	Dimensions	A (in.^{2})	I (in.^{4})
Lower Chord (full)	2 - 10" x 5"	100	833.33
Upper Chord	5.75" x 9.75"	56	154.46
Posts (mid-span)	8" x 10"	80	426.67
Posts (near ends)	11.5" x 10"	115	1267.40
Kingpost	10" x 10"	100	833.33
Diagonal Brace	6.75" x 8"	54	205.03
Double Arch Rib	2 - 5" x 10"	100	416.70

[28] Allen, *Covered Bridges of the Middle Atlantic States*, 48.

Table 2: Material Properties of Eastern White Pine

Modulus of Elasticity, E[29] (psi)	Poisson's Ratio, ν[30]	Density[31] (pcf)
1,100,000	0.4	22

5.2 Condition of Gilpin's Falls Bridge prior to Rehabilitation

When bridgewrights began work on the bridge in May 2009, collapse was imminent. The lower chord and lower arch had failed at the northeast corner of the bridge (Figure 14); the arch ends exhibited severe decay, prohibiting a firm connection with the abutments (Figure 15); and several tension splices in the lower chord had failed (Figure 16). As a result of these failures, the bridge exhibited a 1'-8" lateral bow towards the west (Figure 17).

Figure 14: Northeast corner of Gilpin's Falls Covered Bridge in May 2009. Failure of lower arch and lower truss chord can be readily seen here. Temporary support of the first interior post and lower chord prevented further collapse. Photograph by Rachel H. Sangree.

[29] American Forest & Paper Association, Inc. (AFPA), *NDS National Design Specification, Supplement: Design Values for Wood Construction* (Washington DC, 2006).
[30] Jozsef Bodig and Benjamin A. Jayne, *Mechanics of Wood and Wood Composites* (Malabar, Florida: Krieger Publishing Company, 1993), Table 3.4. Wood is an orthotropic material with six Poisson's Ratios. For a softwood, the average of ν_{LR} and ν_{LT} is approximately 0.4.
[31] Forest Products Laboratory (FPL), *Wood Handbook: Wood as an Engineering Material* (Madison, Wisconsin: United States Department of Agriculture Forest Service, 2010), Table 4-6b.

Figure 15: Southeast corner of bridge exhibited decay of arch ends and end post. Photograph by Rachel H. Sangree.

Figure 16: Typical shear failure of a lower chord / fish plate splice on the east truss. Photograph by Rachel H. Sangree.

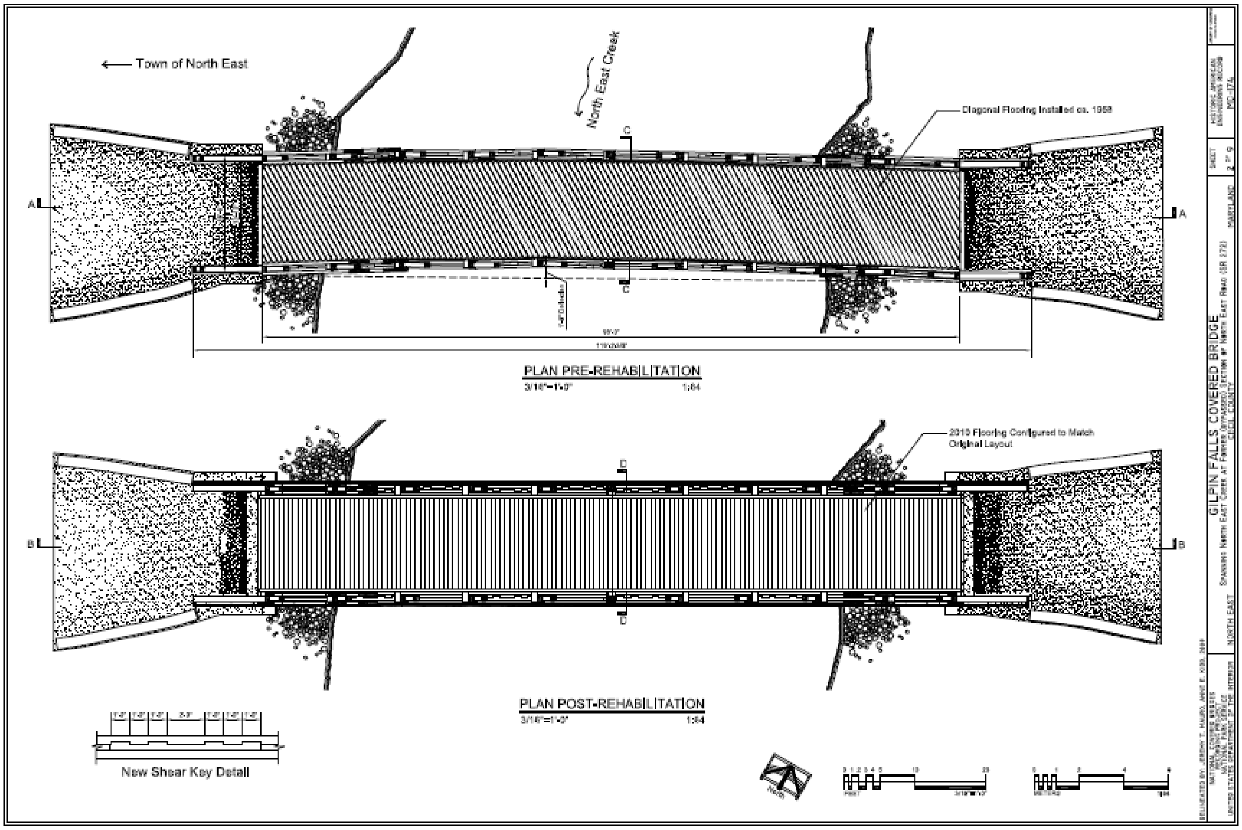

Figure 17: Jeremy T. Mauro and Anne E. Kidd, delineators, Gilpin's Falls Covered Bridge, HAER No. MD-174, Sheet 2 of 9, 2009. Note lateral bow prior to rehabilitation (top plan).

These failures may all be attributed to a combination of wood decay (which caused a loss of support conditions) and connection detailing in the lower chord. A functioning Burr truss requires a smaller, lower chord and less robust lower chord scarf joints than a multiple kingpost acting alone would allow. Additionally, the engagement of the arch with the abutment causes the lower chord in the exterior panel(s) to be compressed, allowing engineers in 1959 to put a butt joint at that location in the lower chord rather than a tension splice. When the ends of the arch deteriorated, however, the thrust (horizontal force at the ends of the arch) could not be resisted by the abutments because there was not enough contact between them and the arch ends (i.e. loss of support conditions). Thus, the thrust was directed into the lower chord, significantly increasing the tensile force in that member. While the member itself (two 5.5" w x 10" d members) could carry the extra force, the splice joints could not. Further, without contact between the arch and the abutment, the compression in the lower chord end panel no longer existed, allowing the butt joint to open up under the additional lower chord force. Eventually the force in the connection between the lower chord and the only remaining effective arch leaf, whose cross-section was reduced to accommodate its intersection with the lower chord, caused the arch to fail.

5.3 2009 Bridge Rehabilitation

As described earlier in the HAER report, rehabilitation of Gilpin's Falls Covered Bridge began in the summer of 2009 by building a temporary steel support structure under the existing truss and using cribbing on top of the temporary structure to support the truss at each post (Figure 18). The siding and floor (with the exception of the floor beam at midspan) and roof systems (with the exception of the tie beams connecting the two trusses) were removed, and the posts were raised enough by the cribbing to remove all axial forces from the truss members. With forces out of the trusses, bridgewrights "walked" the trusses back to vertical, maintaining stability by triangulating the trusses with cross-chains (Figure 19). At this point, members exhibiting damage or decay could be removed (Figure 20).

The lower chord had stretched and sagged as a result of the tension splice failures, so the bridgewrights introduced tie rods into the truss, thus re-establishing the camber and proper geometry of the bridge (drawing the lower chord back to its original length). Once camber was established, the tie rods were removed and the posts were left in their proper position, which the cribbing maintained. Where necessary, the top chord, posts and diagonals were replaced. Once the truss (with the exception of the lower chord and arches) was complete, the tie rods were re-installed and tightened with the purpose of prestressing the top chord and diagonals to increase the truss's stiffness and to better maintain camber. While the tie rods maintained the new prestressed geometry of the truss, the lower chord was rebuilt with stronger scarf joints and installed with 10" of camber (see Figure 21 and Section 9.2).

Figure 18: View from under the bridge where cribbing supporting the six posts closest to the north abutment can be seen. Note: cribbing sits on the floor of the temporary structure, which spans North East Creek. Photograph by Rachel H. Sangree.

Figure 19: Failure of the east truss caused significant lateral movement of the entire structure. Here, the builders have used cross chains to help bring the trusses back to their vertical position. Photograph by Rachel H. Sangree.

Figure 20: Photograph taken at the northeast corner of the bridge, where lower arch and lower chord ends were removed after the bridge was stabilized. Photograph by Rachel H. Sangree.

Figure 21: East truss with newly-installed lower chord and tie rod still in place. Note: the tie rod was removed shortly after this photo was taken, as it was no longer needed. Photograph by Rachel H. Sangree.

At this point the truss was stiff enough to maintain its original 10" of camber, even if the cribbing had been removed. Cribbing remained in place, though, while additional dead load demands from the floor system were placed on the truss. After running some small tests to ensure the strength and stability of the truss under increasing dead load, cribbing was removed and the truss stood, without assistance from the arch.

The truss continued to act independently of the arch, supported at L1 and L9 (see Figure 13), while increasing amounts of dead load were added. Time and weight caused the truss to settle further into its new joinery, becoming stiffer in preparation for the eventual addition of the arch. Starting at midspan and working out toward the abutments, the builders precisely fit adjacent arch-leaves together, precompressing each piece as it was installed and taking care to ensure that the ends of the leaves would bear against each other fully (a joint with a cut just a little out of perpendicular can precipitate the lateral movement of an arch). The arches were reconnected to the truss with their original $^5/_8$"-diameter wrought-iron through-bolts (see Figure 22). The bridge remained in this state, essentially a tied arch, while the concrete for the abutments was poured.[32]

Once the new concrete abutments had cured, the arches were engaged by driving black locust folding wedges between arch ends and abutment faces (see Figure 23). At this point, structural work on the bridge was complete, and builders finished adding the siding and roofing.[33]

[32] Some original wrought-iron rods were replaced with $^3/_4$"-diameter steel rods.
[33] Black locust wood is rot-resistant and used in locations where wood bears against concrete.

Figure 22: Installation of arch segments on the west truss. Photograph by Rachel H. Sangree.

Figure 23: Northeast corner of bridge showing arches engaged with the abutment face via black locust wedges. Photograph by Rachel H. Sangree.

6. EXPERIMENTAL TESTS

6.1 Objectives

As rehabilitation progressed, the bridge's response to increasing dead loads and changing support conditions was monitored with the use of strain gages. This allowed the authors to observe the distribution of dead load among truss and arch members and how that distribution changed as rehabilitation progressed. Monitoring dead load effects represented the thrust of the experimental program, but live load effects were measured as well to gain a more complete picture of the bridge's behavior.

6.2 Dead Load Measurements

Strain gages were glued to the four main "moment-resisting" members near midspan: upper chord, lower chord, upper arch and lower arch (see Figure 24). To capture any out-of-plane strains and to provide for redundant strain measurements, strain gages were glued to the inside and outside face of each member. In all, a total of eight strain readings were recorded at a time.[34]

Strain gages were type N2A-06-20CBW-350, which were used because they were long enough to capture the average strain in a member over a 2" length. A strain gage is shown in Figure 25 (note: the dark region surrounding the gage is the base layer of epoxy used to create a smooth, bondable surface). Axial strains (ε) were recorded with National Instruments LabVIEW software and converted into axial forces (P) by the following expression:

$$P = \varepsilon E A$$

where E is the modulus of elasticity for wood (see Table 2) and A is the cross-sectional area of the truss member (see Table 1). Note that the measured strains resulted not only from axial forces, but also from viscous effects and atmospheric changes.

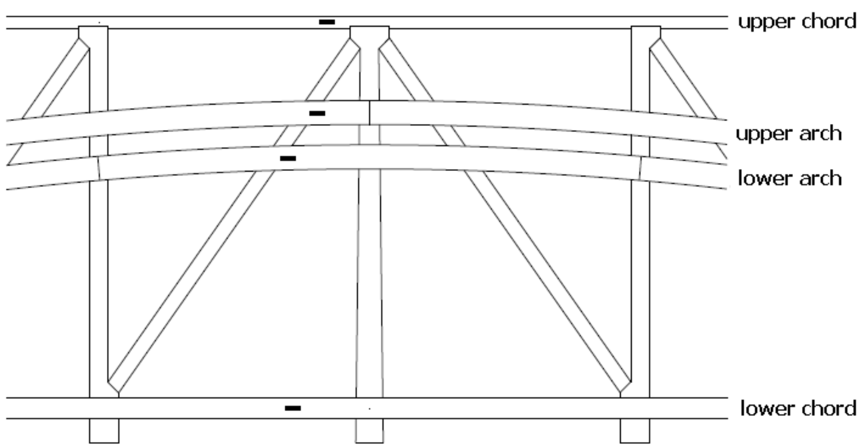

Figure 24: Strain gage locations (▬) just north of midspan on inside face of east truss

[34] Only six of the strain gages provided data throughout the experimental program; the authors observed damage (separation of solder tabs from the plastic backing) to the gage on the inside face of the lower arch and on the outside face of the lower chord after the early stages of rehabilitation. In the case of the lower chord, especially, this presented a problem as the strain gages were glued in the vicinity of a scarf joint, and the gage that was damaged was the one on the continuous length of chord.

Figure 25: Strain gage on inside face of top chord. Photograph by Rachel Sangree.

6.2.1 Rehabilitation Stages

Strain data was recorded at three critical stages within the rehabilitation process. Following is a description of those stages, which occurred during the latter part of the whole rehabilitation process described in Section 5.3.

- ***Stage 1***: The multiple kingpost truss is completed between Post 1 and Post 9. Shoring is removed from under the interior posts (L2 – L8) and the truss supports its own weight in addition to some dead load from the roof and floor systems. Strain data is recorded before and during the removal of shoring.
- ***Stage 2***: The arches are constructed in two-panel leaves and connected to the truss at each post. The timber framers start at midspan and work out towards the supports, ending with the connection to the lower chord. The abutment is not yet complete so the arch thrust directly loads the lower chord through the arch-chord bolted connection. This creates, in effect, a tied arch structure. A single strain reading is recorded during Rehabilitation Stage 2.
- ***Stage 3***: The abutments are completed and the arches are engaged by hammering black locust folding wedges between the arch ends and abutment faces (Figure 23). Though additional siding and roofing is still required, at this point, the Burr truss structure is complete. A single strain reading is recorded during Rehabilitation Stage 3.

The loads present during each rehabilitation stage are provided in Table 3. Note that not all loads are applied to panel points. The loads from the roof, for example, are supported by the rafters, which are in turn supported by the upper chord at an approximate 2'-0" spacing. The siding is supported by nailers that connect to the posts between the top and bottom chords.

Table 3: Panel point and other nodal forces associated with each stage of rehabilitation (see Section 6.2.1)

Panel Point	Stage 1	Stage 2	Stage 3
L0	225	645	645
L1	249	1088	1088
L2	480	1088	1088
L3	711	1088	1088
L4	711	1168	1168
L5	792	1168	1168
L6	711	1088	1088
L7	711	1088	1088
L8	480	1088	1088
L9	249	1088	1088
L10	225	645	645
U0	241	264	264
U1	241	288	288
U2	241	288	288
U3	241	288	288
U4	264	288	288
U5	288	288	288
U6	264	288	288
U7	241	288	288
U8	241	288	288
U9	241	288	288
U_{10}	241	264	264
Other Load Locations	**Stage 1**	**Stage 2**	**Stage 3**
upper chord / rafter	0	21	47
interior post / nailer	0	24	34
end post / nailer	0	12	17

6.3 Live Load Measurements

Today, Gilpin's Falls Covered Bridge carries only pedestrian live load. In order to learn more about the bridge's response to concentrated forces, live load tests were performed by driving a vehicle across the bridge and measuring the changes in member strain and the bridge's global (vertical) displacement. Position transducers (Figure 26) connected between a post and ground (temporary structure) and between the arch and ground measured the global displacement.

Two tests were performed after Rehabilitation Stage 3 had been completed. In the first, position transducers measured the vertical displacement of the kingpost and the lower arch at midspan as a truck was driven across the bridge, stopping so that its center of gravity was first at midspan and then at the three-quarter point (see Figure 27). In the second test, one position transducer was moved to the three-quarter point of the span so that displacement measurements were now for the kingpost at midspan and the arch at the three-quarter point.

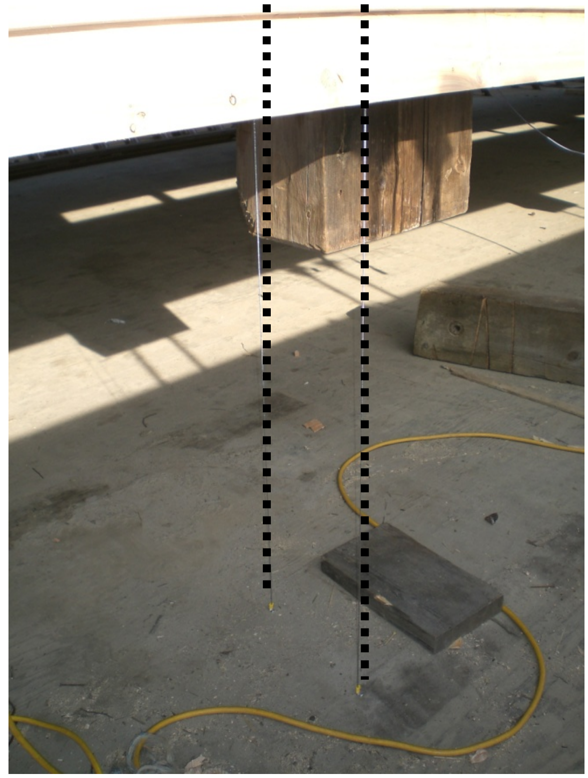

Figure 26: Position transducers were connected to the kingpost and lower arch on the east truss at midspan (wires used to connect the position transducers to ground are highlighted for clarity). Photographs by Rachel H. Sangree.

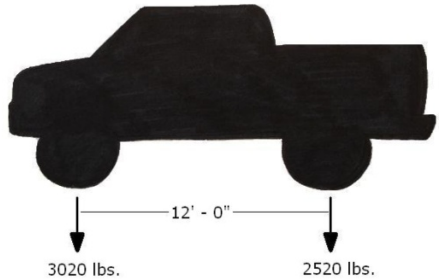

Figure 27: Test vehicle axle weights.

7. NUMERICAL STRUCTURAL MODELS

7.1 Objectives

Numerical structural models are often used to analyze a structure when experimental tests are neither practical nor cost-effective. Even the most carefully modeled structure, however, can predict forces and displacements that are inconsistent with the behavior of the real structure. This is especially true in the case of a traditional timber-framed structure, one whose geometry and support conditions are not as exact as a computer model assumes. In the case of Gilpin's Falls Covered Bridge, the experimental strains offered a means to validate the numerical models.

Two-dimensional linear elastic frame models were created to reproduce, as closely as possible, the loads and boundary conditions on the bridge at each stage of rehabilitation (see Section 6.2.1), and also during the live load tests. This allowed the authors to make comparisons between the two methods – experimental and numerical – later on. The models are described in the following sections.

7.2 Geometry, Section Properties, and Material Properties

MASTAN2 structural analysis software was used to build and analyze the two-dimensional frame models. Geometry was obtained from a previous engineering study performed on the bridge and confirmed or modified as necessary with information obtained at the site.[35] Figure 28 is a model of the bridge in its fully rehabilitated form (Stage 3). The elements in the model represent member centerlines; connecting nodes (●) are located at the intersection of two truss elements or at a location of interest, such as a tension splice. A frame analysis treats all elements as being rigidly connected at nodal locations; thus pinned connections (○) were defined where rotation was allowed to occur. The model also reflects the 10" of camber built into the chords. See Figure 13 for bridge geometry and Table 1 for member section properties. Replacement members used in the rehabilitation matched its original Eastern white pine; material properties used in the frame model are in Table 2.

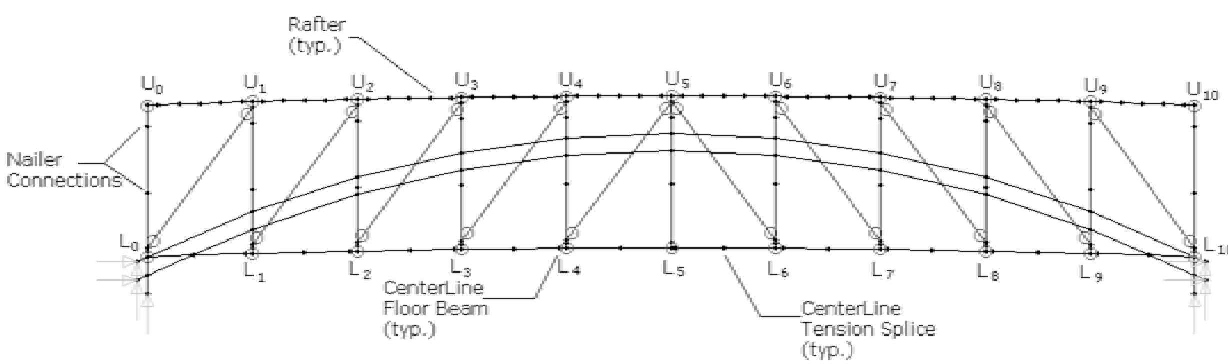

Figure 28: Gilpin's Falls Covered Bridge two-dimensional frame model (completed structure).

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[35] MASTAN2, version 3.2.0, developed by R. D. Ziemian and W. McGuire, 2008. This software is based on the stiffness method of analysis, as described in *Matrix Structural Analysis*. Northeast Engineering, Inc., *Calculations for a Temporary Support Structure for Construction*, 2003.

7.3 Loads and Boundary Conditions

Loads and boundary conditions built into the models varied to simulate the conditions at the bridge for each stage of rehabilitation. Loads were provided previously in Table 3; additional details related to individual models are discussed in Section 7.4 through Section 7.7.

7.4 Stage 1 Frame Model

The Stage 1 frame model is shown in Figure 29. At this point, arch segments were loosely connected to the truss and not providing any strength, so they were left out of the model for simplicity. When the shoring was released the bridge was supported by cribbing at L1 and L9, as indicated by the roller supports below those posts in the model. While the cribbing resisted some displacement in the x-direction through friction with the bottom of the posts, the x-direction was fixed under L5 instead in the model, allowing for a symmetric bridge response and eliminating the development of compressive forces in the lower chord. At this point, the roof load was composed only of the tie beams, which are located at the post-to-upper chord intersection so concentrated forces along the upper chord are applied only at the panel points. The floor system transfers dead load to the truss through a concentrated force at the center line of each floor beam (see Figure 28).

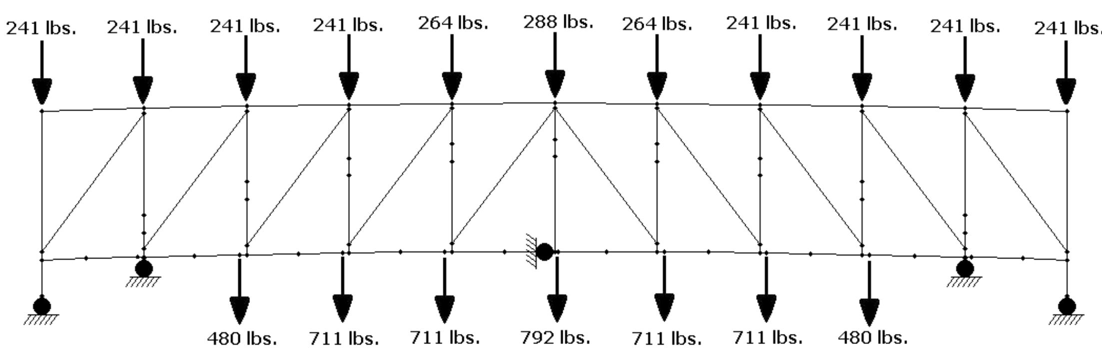


Figure 29: Frame model representing the geometry, loads, and boundary conditions on the bridge during Stage 1 of rehabilitation: the multiple kingpost truss supports its own self-weight between Posts 1 and 9.

7.5 Stage 2 Frame Model

The Stage 2 frame model (Figure 30) represents the second stage of rehabilitation, when the arch was connected to the truss but not yet supported by the abutments. Dead loads on the truss at this stage were significantly higher than in the previous model as builders had continued to work on the floor and roof systems after shoring was removed (see Table 3). The addition of roof rafters meant that the upper chord was loaded transversely between panel points. Posts 0 and 10 were now complete, but builders continued to leave the cribbing under Posts 1 and 9, providing support in the y-direction. Correspondingly, the truss model is fixed in the y-direction at L0, L1, L9, and L10 and in the x-direction at L5, again to allow the model to respond symmetrically.

The self-weight of the truss members as well as some of the weight from the floor system was introduced during Stage 1, requiring the multiple kingpost truss to support that weight without the assistance of the arch. Thus, the Stage 2 model saw only loads applied that were not present when the cribbing was removed, including deck, roof and a small amount of siding. To find the total axial forces in the bridge at Stage 2, the axial forces generated from the Stage 1 model were added to those from the Stage 2 model.

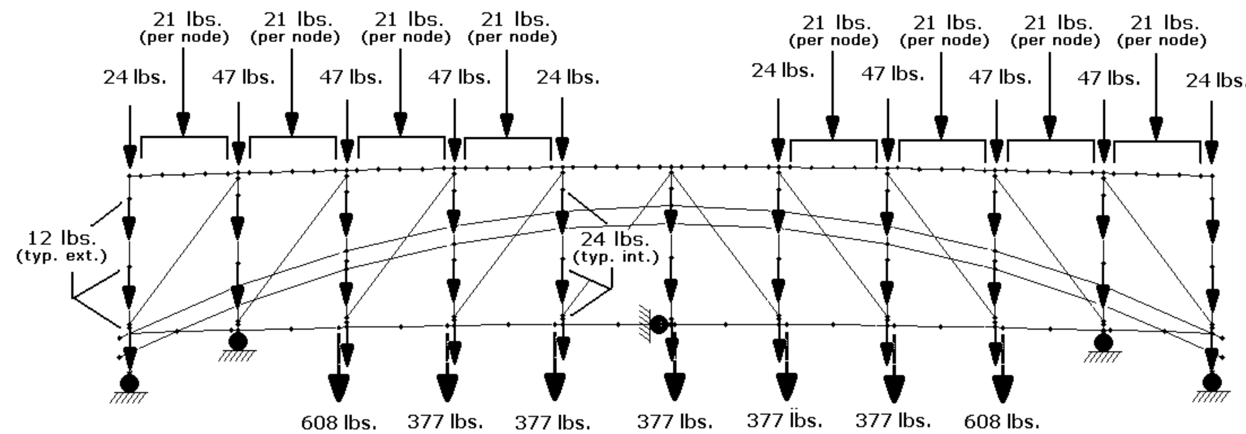


Figure 30: Frame model representing the changes to geometry, loads, and boundary conditions on the bridge during Stage 2 of rehabilitation: the arch is connected to the self-supporting truss.

7.6 Stage 3 Frame Model

In the third stage of rehabilitation, the arches, which were connected to the truss in Stage 2, were engaged with the abutments, again changing the boundary conditions of the model (Figure 31). Now, the truss was supported in the y-direction at L0 and L10, and the arches were pinned at both ends, reflecting the connection to the abutments via the folding wedges. Although the placement of the wedges would have introduced some compressive force into the bridge, it was ignored in the model because the quantity of this force was not known. Again, only loads from the roof and siding added after Stage 2 were considered in the Stage 3 model. The weight of the siding present at this stage was applied to the truss at the nailer locations (see Figure 28).

Stage 3 presented a modeling challenge in that the axial forces due to self-weight of the kingpost truss alone (found from the Stage 1 model) would have been redistributed in Stage 3 when the supports at L1 and L9 were removed, thus increasing the span length. Further, floor loads from Stage 1 that would have been applied to L1 and L9, had they not been directly supported by those supports, were left out of the Stage 1 analysis and needed to be considered.

Figure 32 demonstrates how these two items were handled in the Stage 3 numerical analysis. Two additional models were created. In the first, Figure 32a, an analysis was performed on the Stage 3 model considering only the self-weight of the truss members, and the support conditions as they existed prior to the removal of L1 and L9. In the second, Figure 32b, an analysis was performed on the Stage 3 model considering both the self-weight of the truss members and the floor load applied to L1 and L9, with the supports at L1 and L9 removed. The results of the second model were subtracted from the first model to quantify the effect on existing axial forces of changing support conditions. The method was not exact, but it did allow for some quantification of this change.

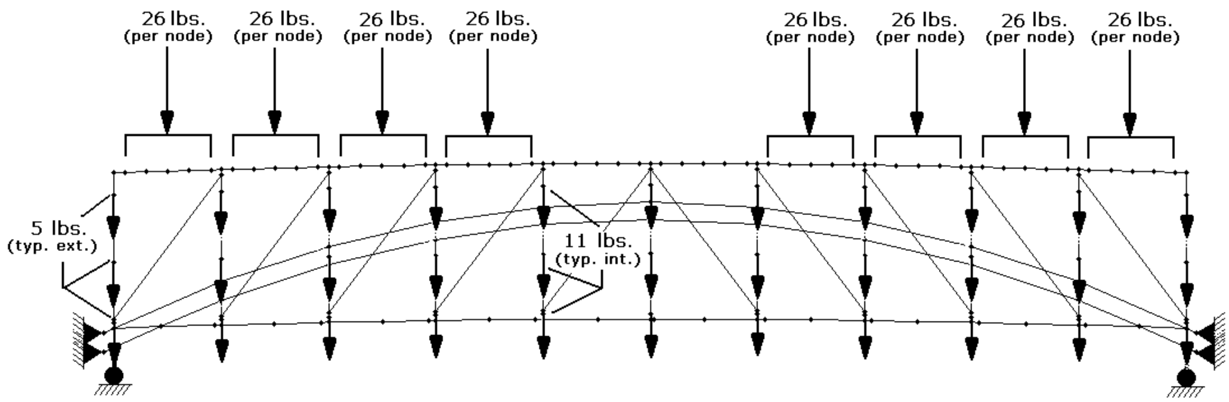
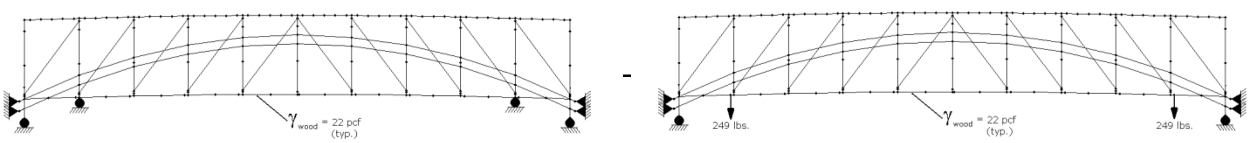


Figure 31: Frame model representing changes to geometry, loads, and boundary conditions on the bridge during Stage 3 of rehabilitation: the arch is engaged with the abutments.



(a) (b)

Figure 32: Additional axial forces were generated during Stage 3 due to the removal of supports at L1 and L9. The difference in results of two models was used to quantify these additional axial forces. (a) applies the self-weight of the truss (calculated automatically with the density shown) to the model with L1 and L9 supported; (b) applies the self-weight of the truss to the model with supports and L1 and L9 removed, in addition to the concentrated floor loads that were previously supported directly by the cribbing.

7.7 Live Load Frame Model

Live load tests were performed on the bridge during Stage 3 of rehabilitation; thus the boundary conditions were identical to those in Section 0, but the dead loads were removed. The weight of the truck axles is given in Figure 27 and a simple distribution to the floor beams based on the truck axles' positions was assumed for analysis purposes. The live load distribution for the truck at midspan is shown in Figure 33 and for the truck at the three-quarter point in Figure 34.

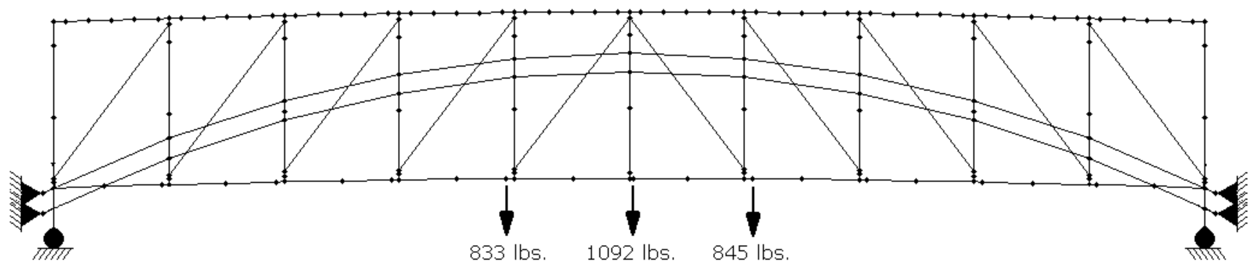


Figure 33: Two-dimensional frame model for live load at midspan.

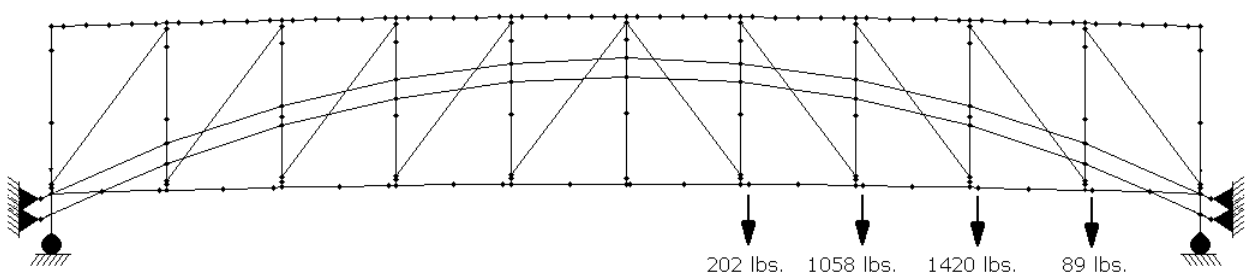


Figure 34: Two-dimensional frame model for live load at the three-quarter point.

8. COMPARISON OF RESULTS

8.1 Rehabilitation Stage 1

Rehabilitation Stage 1 refers to the initial removal of shoring from under the bridge. This was the first point during rehabilitation when the bridge supported its own self-weight. Strain in the four instrumented members was measured before and during the shoring removal, offering a dynamic view of the distribution of load among the bridge members. These results are presented in Figure 35.

The midspan axial forces are consistent with a kingpost truss supporting its own weight. The axial forces in the upper and lower chords are nearly equal in magnitude, but opposite in direction, demonstrating the formation of a force couple in the chords at midspan to resist the external moment. While portions of the arches were loosely connected to the truss at this point, they did not support a significant amount of load, confirming the applicability of the Stage 1 numerical model which ignores the arches altogether.[36]

[36] A negative value indicates compression; a positive value indicates tension

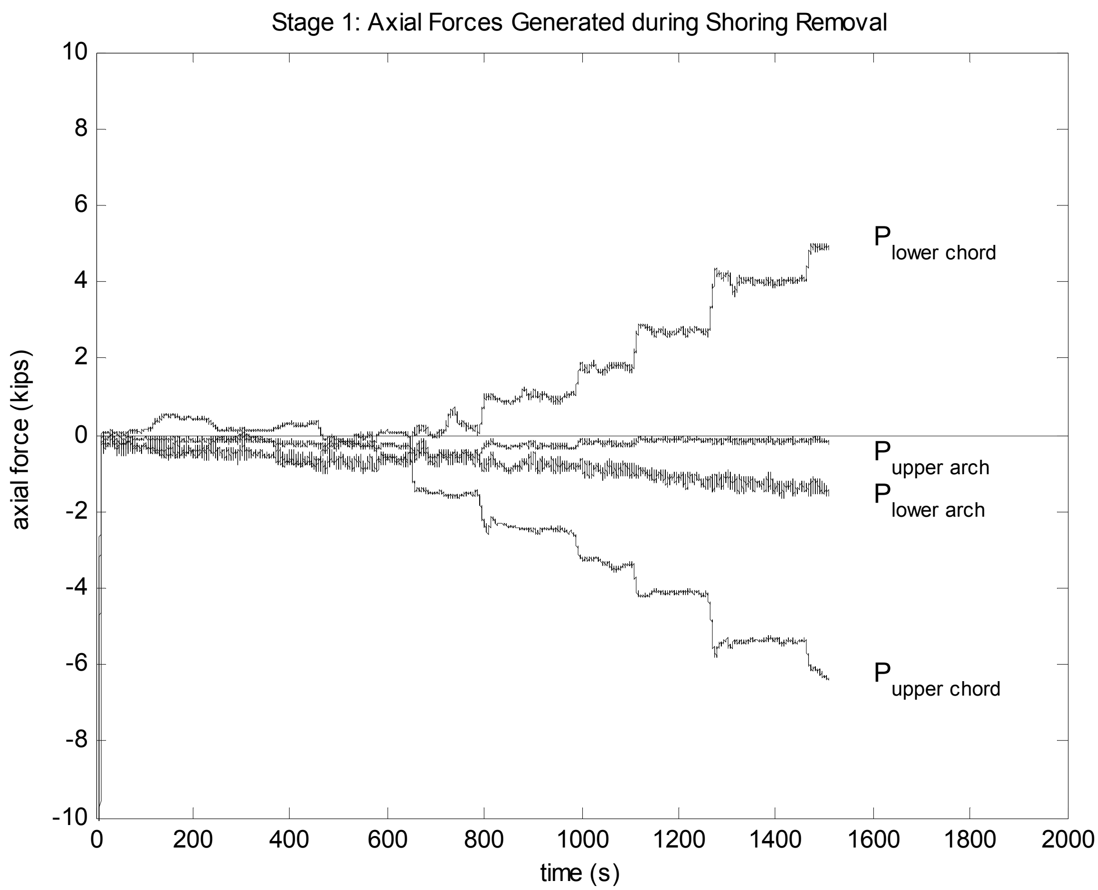


Figure 35: Axial forces generated in chords and arches at midspan as shoring was released.

The axial force diagram resulting from the numerical model is shown in Figure 36, which demonstrates all the characteristics of a truss subjected to a uniform load, with nearly equal tension and compression in the upper and lower chords, respectively (see Table 4). These numerical predictions are presented together with the experimental data in Table 4. While good agreement exists for compression in the upper chord, the tension in the lower chord was predicted to be 46 percent larger than what was measured. This may be explained by the boundary conditions at L1 and L9 in the numerical model, which allowed translation; in reality, the cribbing at L1 and L9 provided some resistance to longitudinal displacement, thereby decreasing the tension force in the lower chord.

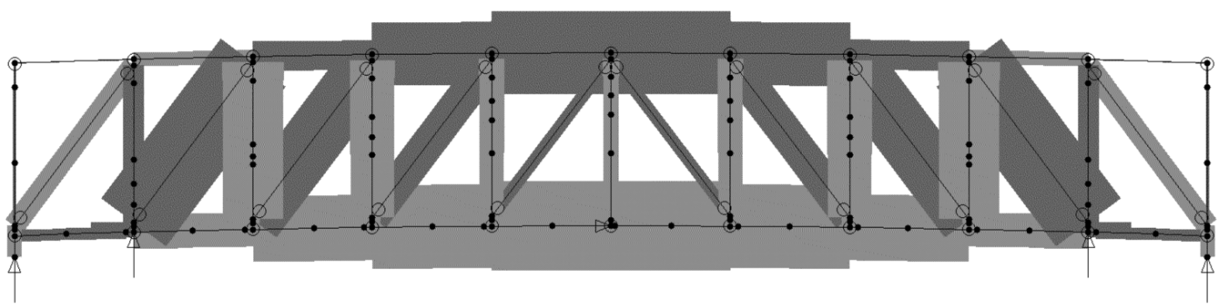


Figure 36: Axial force diagram for Rehabilitation Stage 1 (Note: shaded blocks superimposed on the truss members represent axial force magnitudes; dark shading indicates compression and light shading indicates tension).

Table 4: Axial forces in the upper and lower chords resulting from shoring removal in Stage 1; Note: this data accompanies Figure 37

Member	**Stage 1 (removal of shoring) Axial Forces (kips)**	
	experimental data	*numerical prediction*
upper chord (y = 168")	-6.39	-6.67
lower chord (y = 0")	4.99	7.31

The experimental and numerical forces from Table 4 may also be visualized graphically as in Figure 37, which plots the chord forces as a function of each member's height above the lower chord at midspan. The reader may consider the vertical line on this plot to represent the kingpost in the truss. The experimental data from Table 4 are shown as gray bars and the numerical predictions are plotted as points. Note that in both numerical predictions and experimental results, a positive axial force corresponds to axial tension and a negative axial force corresponds to axial compression. Later plots will include two additional sets of data for the upper arch (y = 128") and lower arch (y = 108"), but at this stage of construction the arch did not yet play a significant role.

Not included is one additional measurement recorded by the bridgewrights during the first stage of rehabilitation: the bridge was surveyed before and after shoring removal and found to displace $^3/_4$" at midspan. The numerical model predicted a displacement of only 0.29" at midspan.

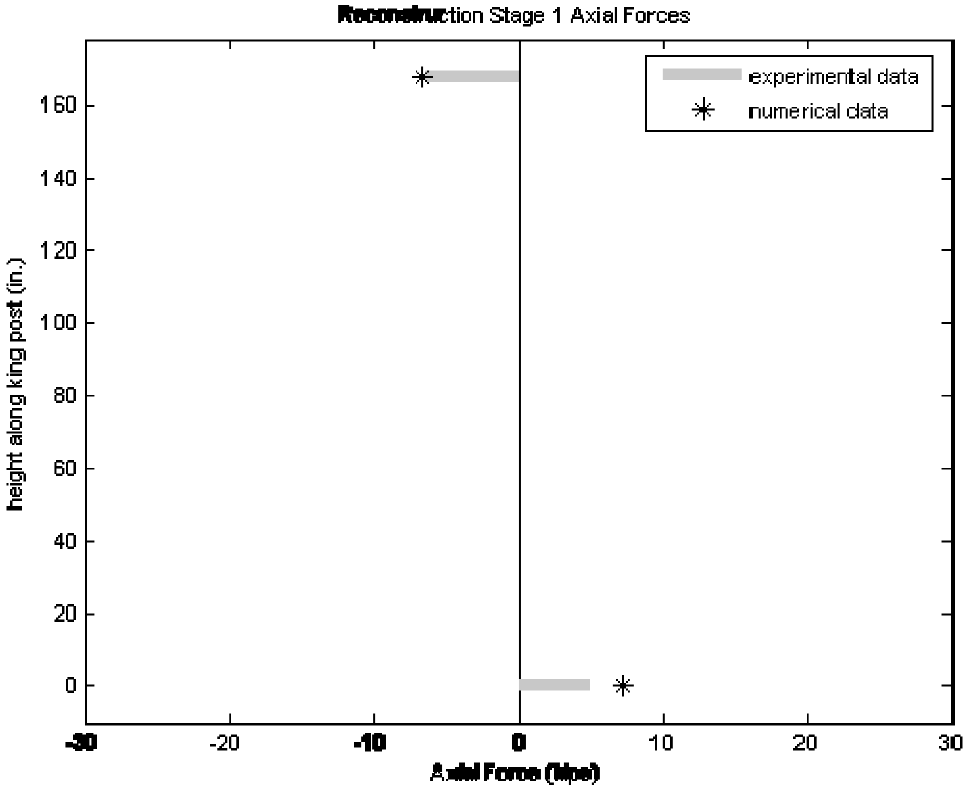


Figure 37: Axial forces in the upper and lower chords at midspan during Rehabilitation Stage 1. Note: negative axial force corresponds to compression; positive axial force corresponds to tension. Thus, this figure shows that the lower chord experienced 4.99 kips axial tension, with a value of 7.31 kips tension predicted by the numerical model. The upper chord, 168" above the lower chord, experienced 6.39 kips axial compression with a value of 6.67 kips compression predicted by the numerical model.

8.2 Rehabilitation Stage 2

In keeping with Daniels' approach to Burr arch construction, Stage 2 saw the addition of the arch to the truss, with a small amount of additional dead load. An important detail to note is that the axial strain measurements were recorded before the arch was engaged with the abutment, so that any load carried by the arch at this point created a thrust (i.e. horizontal force) directed into the lower chord. The numerical model was updated to reflect the changes in geometry, but the dead loads that existed during Stage 1 were not applied, as their effects were already present in the truss members. Instead, the results from the Stage 1 numerical model were added to the results from the Stage 2 numerical model to find the axial forces present in each member at the end of Stage 2. The experimental data and numerical predictions are compared in Figure 38 and Table 5. Note that while the arches were present, the strains in Stage 2 could not stand alone as they had been significantly impacted by construction that occurred between Stage 1 and Stage 2. Thus, arch strains in Stage 2 provided "baseline data" for the arch strains measured in Stage 3 and were assumed to be zero in Stage 2. That being said, a likely explanation for the difference in experimental and numerical axial forces shown in Figure 38 and Table 5 is that as the arch leaves were put in place, they were precompressed with a hydraulic jack, driving additional thrust into the lower chord (thereby increasing the axial tension force) while relieving the upper chord of some of its compressive force.

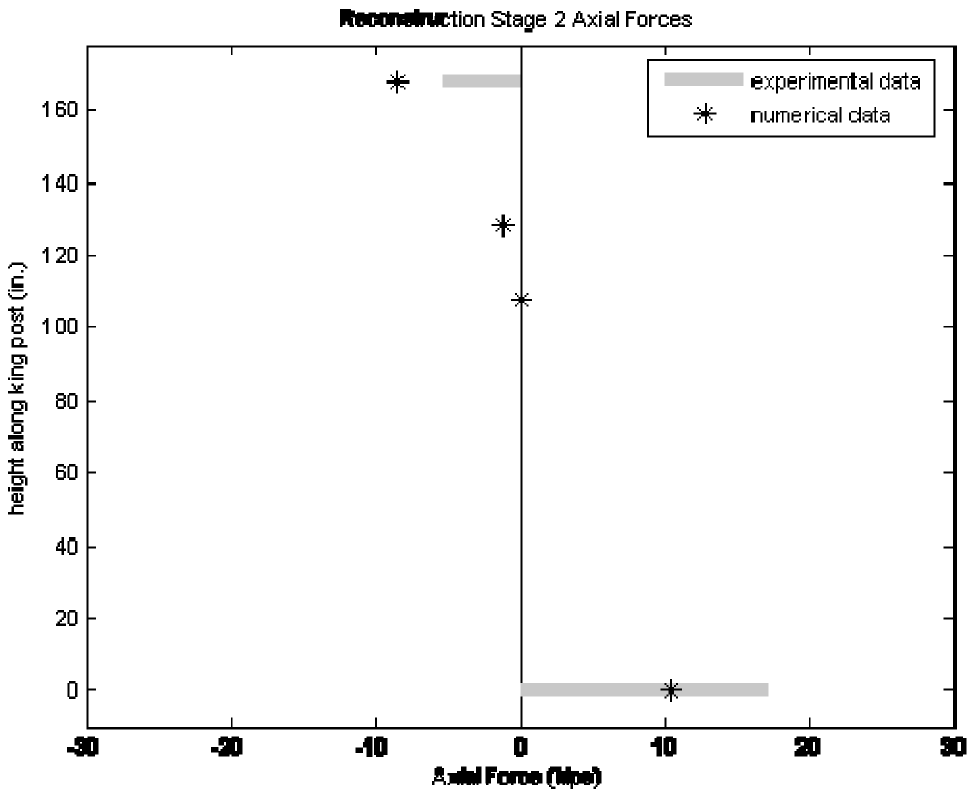


Figure 38: Axial forces in the chords and arches at midspan during Rehabilitation Stage 2

Table 5: Axial forces in the chords and arches at midspan during Rehabilitation Stage 2; Note: this data accompanies Figure 38.

Member	**Stage 2 (introduction of the arch) Axial Forces (kips)**	
	experimental data	*numerical prediction*
upper chord (y = 168")	-5.32	-8.55
upper arch (y = 128")	-	-1.23
lower arch (y = 108")	-	0.13
lower chord (y = 0")	17.5	10.5

8.3 Rehabilitation Stage 3

During Stage 3 of rehabilitation, the last for which data was recorded, the arches were physically engaged with the completed abutments. A comparison of the numerical predictions with experimental data is made in Figure 39 and Table 6. Agreement between forces is good in all but the lower chord. The substantial increase in lower chord axial force from Stage 2 results at least in part from the strain gage on the outside face of the lower chord becoming detached (see footnote in Section 6.2). Both the arches and the lower chord are constructed of two individual members that may share the load differently, but when added together provide the total axial force being supported by the member. Without the other half of the lower chord, this value contains error.

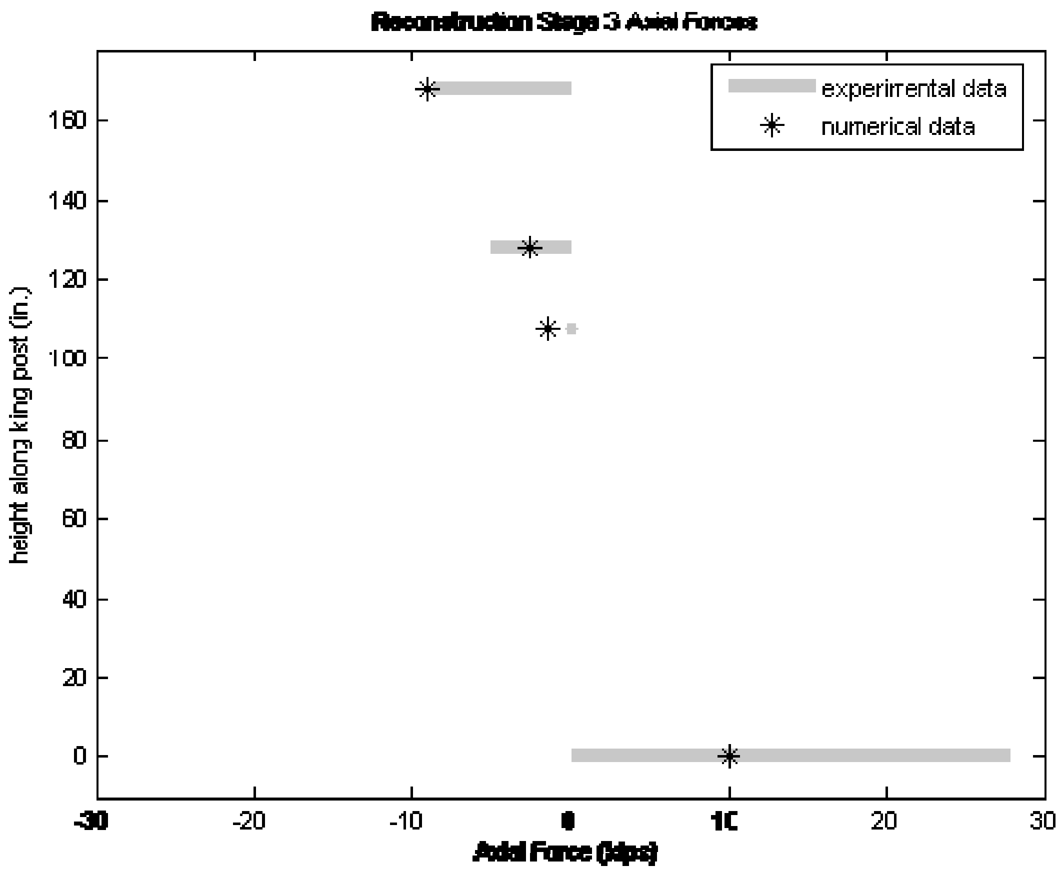


Figure 39: Axial forces in the chords and arches at midspan during Rehabilitation Stage 3

Table 6: Axial forces in the chords and arches at midspan during Rehabilitation Stage 3; Note: this data accompanies Figure .

Member	Stage 3 (engagement of the arches with the abutments) Axial Forces (kips)	
	experimental data	*numerical prediction*
upper chord (y = 168")	-8.60	-9.01
upper arch (y = 128")	-4.90	-2.62
lower arch (y = 108")	-0.96	-1.41
lower chord (y = 0")	28.0	9.99

8.4 Live Load Tests

Researchers performed live load tests by driving a pickup truck across the bridge from north to south and stopping it at midspan and the quarter-point to take member strain and global displacement measurements. The results of these tests produced the influence data shown in Figure 40. The nearly-identical displacements recorded from the arch and truss position transducers demonstrate that the through-bolts used to transfer load from truss to arch provide a stiff and effective connection. Figures 41 and 42 present member axial forces derived from member axial strains for a live load at midspan and at the quarter point, respectively. The error associated with the lower chord axial force may again be attributed to the reliance on only one working strain gage for a two-member chord.

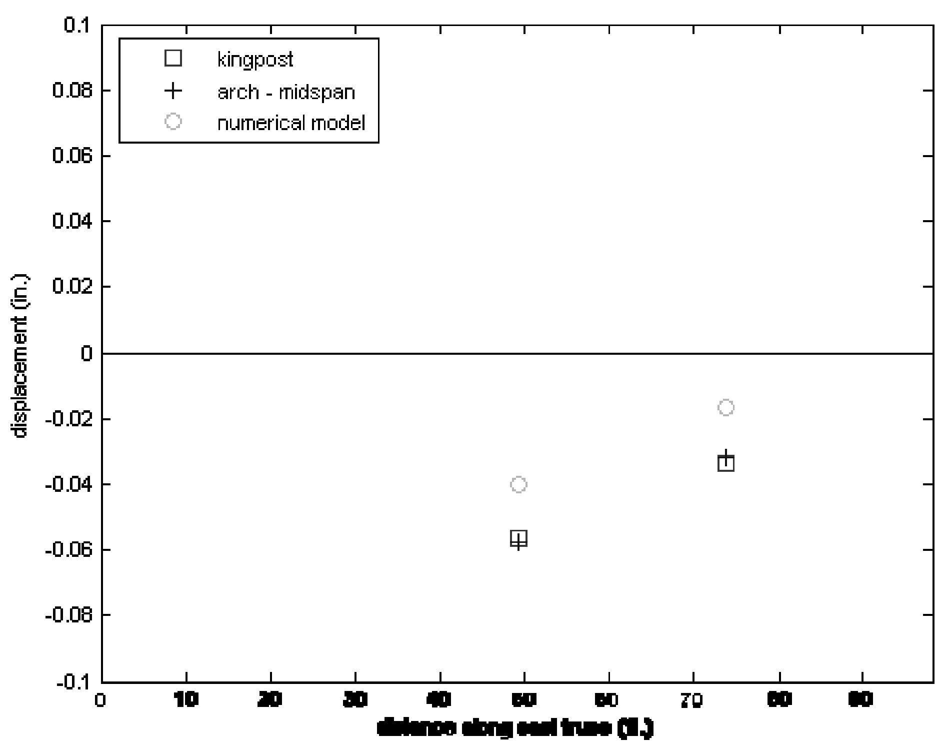


Figure 40: Effect of a moving live load (placed near midspan, 49'-0" and near the quarter point, 75'-0") on displacements at midspan.

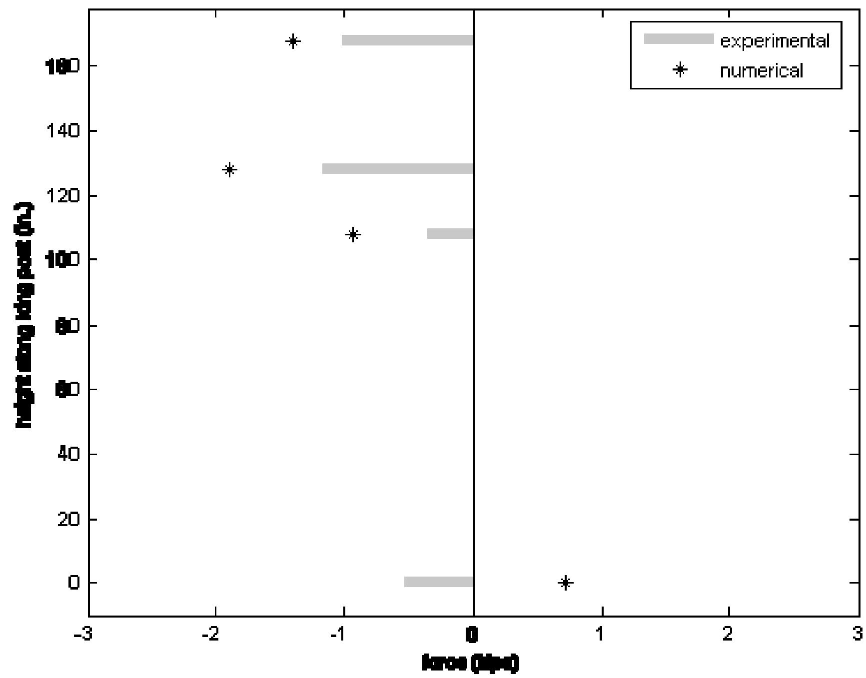


Figure 41: Effect on member axial forces of placing a concentrated live load at midspan.

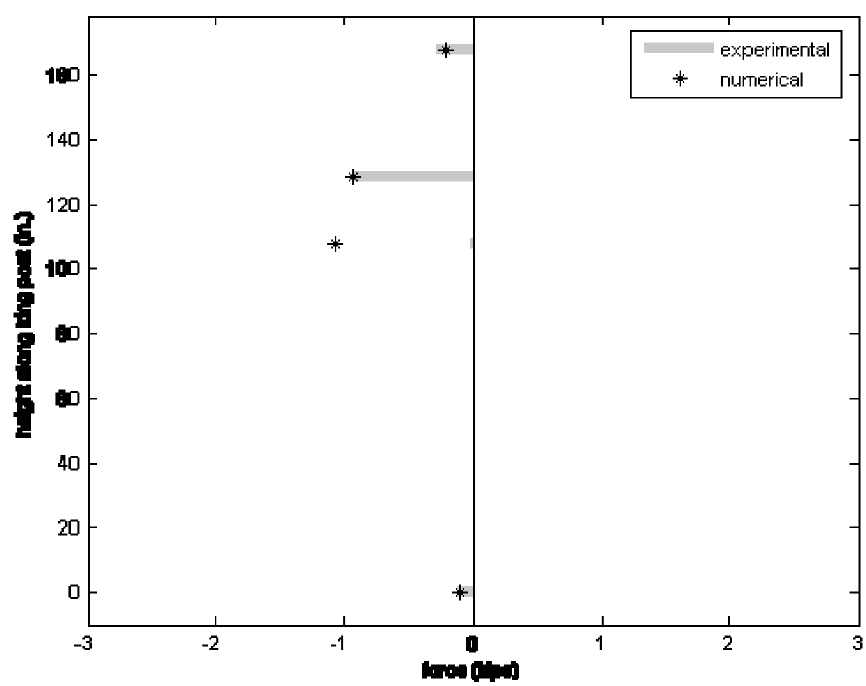


Figure 42: Effect on member axial forces of placing a concentrated live load at the quarter point.

9. DISCUSSION

Further investigation occurs in the following sections on the capacity of each system and its components, and the influence of connection stiffness, construction sequence and long-term creep on the distribution of load between the arch and truss.

9.1 System capacity

It has been suggested that nineteenth-century builders of the Burr truss sized the truss and arch members so that each system was capable of carrying all of the loads without the assistance of the other. Indeed, this was shown to be the case in Barrackville Covered Bridge, and it makes sense that George Johnson would have approached the design of Gilpin's Falls Covered Bridge in a similar manner, given the structural analysis tools available to him in 1860. To investigate the claim further, this section compares member demands within the individual systems under dead load.[37]

The current National Design Specification (NDS) was used to calculate member capacities in moment, shear, axial tension and axial compression; these values are provided in Table 7. NDS notation for design capacity is used in the following analysis: M' (bending moment capacity), V' (shear force capacity), T' (tensile force capacity) and P' (compressive force capacity). [38]

Table 7: Member capacities based on the Allowable Stress Design (ASD) philosophy in the National Design Specification (NDS)

Member	M' (in.-kips)	V' (kips)	T' (kips)	P' (kips)
upper chord	31.3	4.2	23.3	30.3
lower chord	131.0	7.5	31.4	54.0
arch (single)	65.1	3.8	18.7	27.0
post	62.1	6.0	32.8	43.2
diagonal brace	35.3	4.1	18.2	29.2

Table 8 contains the maximum forces (moment, shear, axial force) that would develop in each truss member if the multiple kingpost truss acted alone in supporting the dead load; likewise, Table 9 contains the maximum forces that would develop in the arch if it was acting alone. Again, in keeping with current NDS notation, the member forces (i.e. demands) are denoted as M (bending moment), V (shear force), T (tension force) and P (compressive force). Table 10 facilitates comparisons among Table 7 (member capacity), Table 8 (demands on the truss-only system) and Table 9 (demands on the arch-only system), by showing the ratio of demand:capacity for each member. For example, the upper chord sees a maximum moment of 7.4 inch-kips under the truss-only condition (Table 8), and it has a capacity of 31.3 inch-kips (Table 7); the ratio of demand:capacity is then 7.4/31.3 = 0.24 (Table 10). A value less than unity indicates that demand is less than capacity and a value greater than unity indicates that demand is greater than capacity and thus the member is under-designed.

[37] David Fischetti, *Structural Investigation of Historic Buildings* (Hoboken, New Jersey: John Wiley & Sons, Inc. 2009); Kemp and Hall, "Case Study of a Burr Truss Covered Bridge."
[38] American Forest & Paper Association, *NDS National Design Specification, Supplement*.

Table 8: Member demands resulting from dead loads applied to the multiple kingpost truss model only

Member	M (in.-kips)	V (kips)	T (kips)	P (kips)
upper chord	7.4	0.37	-	-22.4
lower chord	8.7	1.2	24.5	-
post	66.7	8.9	11.2	-
diagonal brace	-	-	-	-15.6

Table 9: Member demands resulting from dead loads applied to the two-hinged parabolic arch model only

Member	M (in.-kips)	V (kips)	T (kips)	P (kips)
arch	6.2	0.32	-	-8.9

Table 10: Ratios of dead load demand to member capacity (note: a value greater than 1.0 is unsafe)

Member	M/M' (in.-kips)	V/V' (kips)	T/T' (kips)	P/P' (kips)
upper chord	0.24	0.088	-	0.74
lower chord	0.066	0.16	0.78	-
arch	0.10	0.08	-	0.33
post	1.07	1.48	0.34	-
diagonal brace	-	-	-	0.53

The axial force ratios are all well below 1.0, indicating that Johnson did account for plenty of reserve strength even in the individual systems. However, the level of reserve strength varies among members. For example, Table 10 demonstrates that while dead load axial demands on the arch account for only 33 percent of its capacity, dead load demands on the upper chord account for 74 percent of its capacity. It is difficult to say with any assurance what the reason for this variation is, but the significant amount of reserve capacity in the arch does suggest that Johnson thought it would be relied upon heavily to carry the bridge loads. According to Table 10, the only members under capacity are the posts (ratio of moment demand:capacity is 1.07 and shear demand:capacity is 1.48), which arises from significant shear and moment demands caused by terminating the diagonal brace between the post ends rather than at a location concentric with the post-to-chord connections. This creates a situation in which the post must transfer axial forces from the diagonal member to the chord via bending rather than axial tension and is a typical detail found on Burr trusses because it was necessary for construction. While this and other studies have found the post-diagonal detail to create a potential area for failure, to the author's knowledge, no failures of a Burr truss have occurred because of it.[39]

[39] Lamar and Schafer, "Structural Analyses of Two Historic Covered Wooden Bridges."

While the redundancy built into the system made the initial sizing of members simpler, it did not provide insurance against a different type of failure, one caused by a loss of arch support. Failure occurred in Gilpin's Falls Covered Bridge not because one system or the other failed, but because the loss of arch support at the abutment created a tied arch structure, wherein the thrust from the arches was directed into the lower chord, significantly increasing the tensile force in that member. Dead load demands generated under these conditions and the ratios of member demand:capacity are reported in Table 11 and Table 12, respectively.

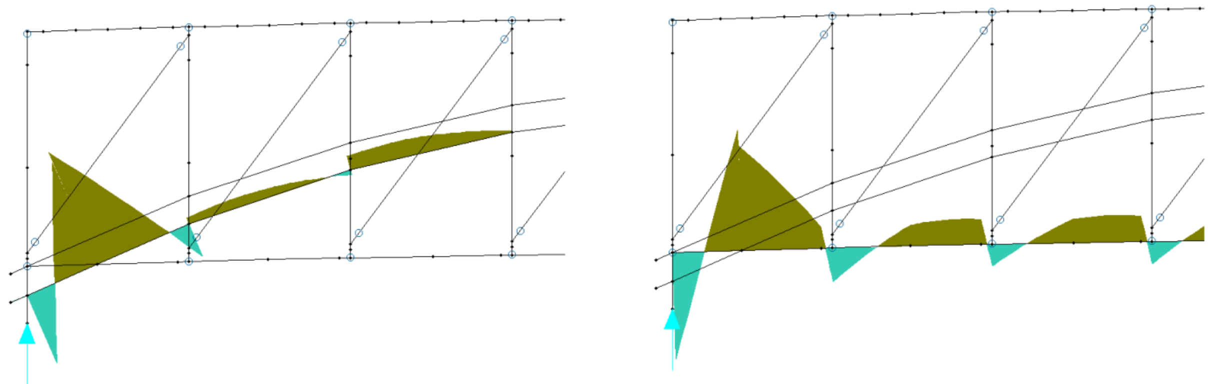
Further validation of the cause of the bridge's failure (see Section 5.2) comes from the moment diagrams generated from analyzing the tied arch system (see Figure 43). Had the arch and lower chord had use of their full sections under such conditions it is possible that they would have had the capacity to support the high bending moments (see Table 12 which assumes the full section is available). However, the lower chord had been retrofitted with a butt joint between L0 and L1, which was now subjected to tension, rendering half of its section useless in resisting the moment. Additionally, the lower arch has a significantly reduced cross-section in this area to accommodate the lower chord connection.

Table 11: Member demands resulting from the analysis of a tied arch model

Member	M (in.-kips)	V (kips)	T (kips)	P (kips)
upper chord	6.7	0.37	-	-14.5
lower chord	23.3	1.36	29.9	-
arch	35.0	0.98	-	-12.2
post	57.7	5.77	7.98	-
diagonal brace	-	-	-	-10.3

Table 12: Ratio of member demands to capacities resulting from the analysis of the tied arch model

Member	M/M' (in.-kips)	V/V' (kips)	T/T' (kips)	P/P' (kips)
upper chord	0.21	0.09	-	0.48
lower chord	0.18	0.18	0.95	-
arch (single)	0.54	0.26	-	0.45
post	0.93	0.96	0.24	-
diagonal brace	-	-	-	0.35



(a) **(b)**

Figure 43: Moments induced in (a) the lower arch and (b) the lower chord resulting from a loss of support conditions at the arch ends.

9.2 Reconstructing the Lower Chord Tension Splice

The tensile capacity of the lower chord provided in Section 9.1 is controlled by the strength of the lower chord tension splice. The particular tension splices used in the lower chord of Gilpin's Falls Covered Bridge are butt joints with wooden fish plates and clamping bolts. A fish plate is a notched length of wood that clamps onto the lower chord and fits into corresponding notches cut into the ends of the lower chord members being spliced (see Figure 44a). The fish plates alternated between the outside and inside chord members along the length of the bridge so a minimum of one chord was continuous at all locations. Clamping bolts were also installed along the length of the splice, not to provide shear strength to the connection but rather to prevent bending of the fish plate due to the eccentric tensile force, which would reduce contact between the opposing notches.[40]

The capacity of this type of tension splice is found by calculating the minimum strength of three conditions that would result in the splice's failure: end grain bearing strength at the notch, shear strength of the notched length and the tensile strength of the net section. Prior to the bridge's rehabilitation, bridgewrights observed that most of the lower chord splice joints had failed, most often in shear (see Figure 44b), though crushing of the wood at the notches as in a bearing failure was calculated as the controlling mode and likely occurred much earlier.

The following calculations, in addition to the observed failure mechanisms discussed above, confirmed that the original lower chord tension splice was undersized. The fish plates were redesigned to include two – 2" deep notches (the original design used one – 1" deep notch) and two – 12" long shear planes (the original design used one – 12" long shear plane), resulting in a tension splice with 30 percent more capacity than the original (see Figure 45 and

[40] Phillip C. Pierce, Robert L. Brungraber, Abba Lichtenstein, Scott Sabol, J.J. Morell and S.T. Lebow, "Covered Bridge Manual," FHWA-HRT-04-098 (Turner-Fairbank Highway Research Center, Federal Highway Administration, U.S. Dept. of Transportation, April 2005), 148.

Table 13).[41] In addition to the original and reconstructed tension splice capacities, Table 13 summarizes the dead load demands on the lower chord resulting from analyses on the multiple kingpost truss acting alone (Stage 1 Construction), the tied arch system (Stage 2 Construction / state of the bridge with deteriorated arch ends) and the Burr truss (Stage 3 Construction). Note that the maximum tensile force in the lower chord occurs when the ends of the arch are not engaged with the abutments, but are rather directing their thrust into the lower chord.

The low capacity of the original lower chord tension splice suggests that even if the builders sized the truss to carry 100 percent of the load without assistance from the arch, they may have reduced the capacity of the splice with the understanding that a well-functioning arch would reduce the tensile force in the lower chord.

(a) (b)

Figure 44: (a) Original lower tension splice in Gilpin's Falls Bridge and (b) typical shear failure of a tension splice (note the movement of the lower chord due to joint failure). Photographs by Rachel H. Sangree.

[41] Capacity referred to is calculated using Allowable Stress Design (ASD) method in the 2005 edition of the National Design Specification for Wood Construction (NDS).

Figure 45: New lower chord tension splice (outlined for visibility). Photograph by Rachel H. Sangree.

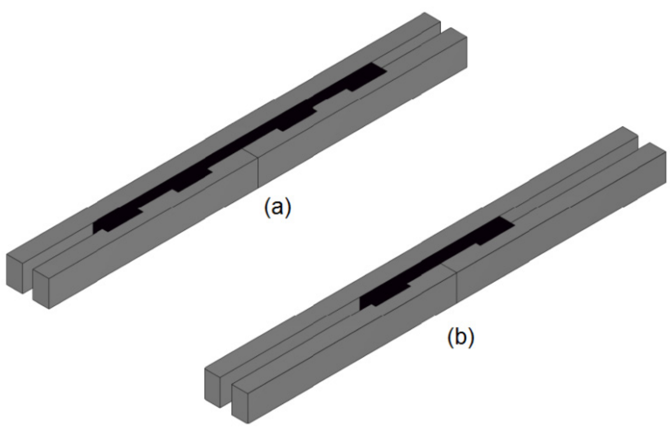


Figure 46: Lower chord tension splices: (a) reconstructed and (b) original designs. Note: lower chord members are shaded gray; fish plates are shaded black.

Table 13: Lower chord capacities and demands under different truss configurations

Lower Chord Capacity (kips)		Lower Chord Demand (kips)		
Original Design	***Rehabilitated Design***	***Multiple Kingpost (Stage 1)***	***"Tied Arch" (Stage 2)***	***Burr Truss (Stage 3)***
31.4	40.93	28.3	37.6	1.40

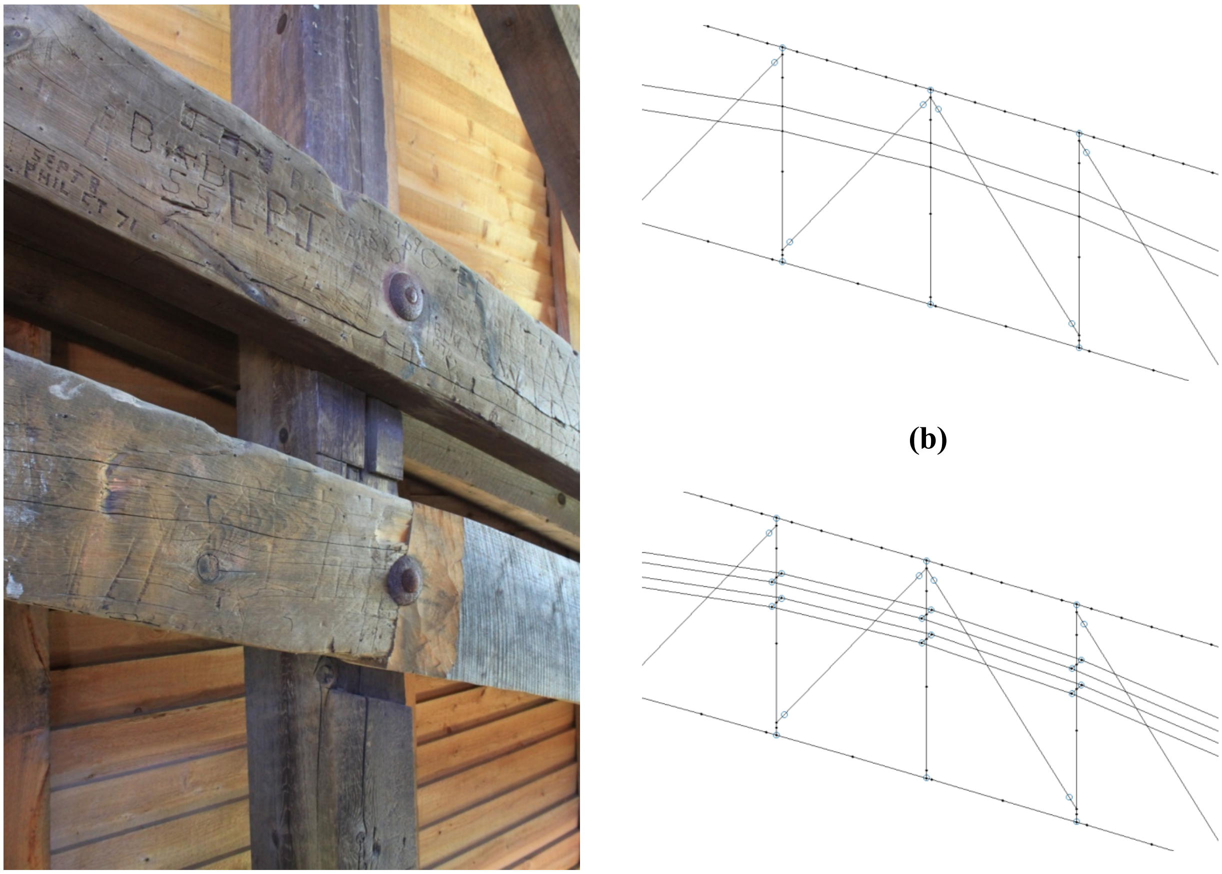
9.3 Influence of Arch-Truss Connection Stiffness

Loads, whether sustained (dead) or temporary (live), are not directly applied to the arch. Dead loads from the floor are transferred to the truss by the floor beam connections to the lower chord; dead loads from the roof are transferred to the truss by the rafters that rest on the top of upper chord. Like dead loads from the floor, live loads also reach the truss via the floor beams. The amount of load that reaches the arch is therefore entirely dependent on the stiffness of the connection between the two systems.

Gilpin's Falls Covered Bridge uses a typical connection between the arch and truss consisting of a $^{5}/_{8}$"-diameter wrought-iron through-bolt passing through the inside arch, truss post and outside arch. In addition, the post is notched $^{1}/_{2}$" on both faces to accommodate the arches (see Figure 47a). The numerical models discussed in Section 7 were two-dimensional, idealizing the truss elements (i.e. arches and truss) as being located in a single plane, when in reality the through-bolt that connects the arch and truss is perpendicular to the plane of the truss, and the arch is offset from the truss. Both the arch and truss are continuous through their connections, but in reality they can rotate about the z-axis with respect to one another. Thus, the single-plane idealization has implications on the joint rotation allowed between the arch and truss.

A space frame model including an element to model the out-of-plane through-bolt was created to examine the effect of changes in connection stiffness between the arch and truss. The arches were separated into their halves and offset from the truss. The through-bolt was modeled as an out-of-plane beam element connecting the two arch-halves to the post. Torsional resistance was eliminated at either end of the through-bolt element to allow the arch and truss to rotate with respect to one another. The plane frame and space frame models of the arch-to-truss connection are shown in Figures 47b and 47c, respectively.

While this connection may become more flexible over time, especially in the case of a bridge exposed to cyclic live loading, the experimental live load test results in Figure 40 demonstrate that post-rehabilitation, the truss and the arch displaced in unison—a perfectly rigid connection as a result of rehabilitation. Over time, however, if the connection between the two systems becomes more flexible, less load might be transferred to the arch from the truss; as the connection stiffness decreases to zero (an unlikely situation unless the threaded rods were removed or failed in shear) the Burr truss would be reduced to a multiple kingpost truss. The dependency of the structural behavior on the connector stiffness is demonstrated qualitatively in Figure 48.



(b)

(a) (c)

Figure 47: Arch-to-post connection (a) as it appears at the bridge (most of the connections looked like the one on top, but in a few locations the arch was not seated in the post notch) (b) in a plane frame model and (c) in a space frame model.

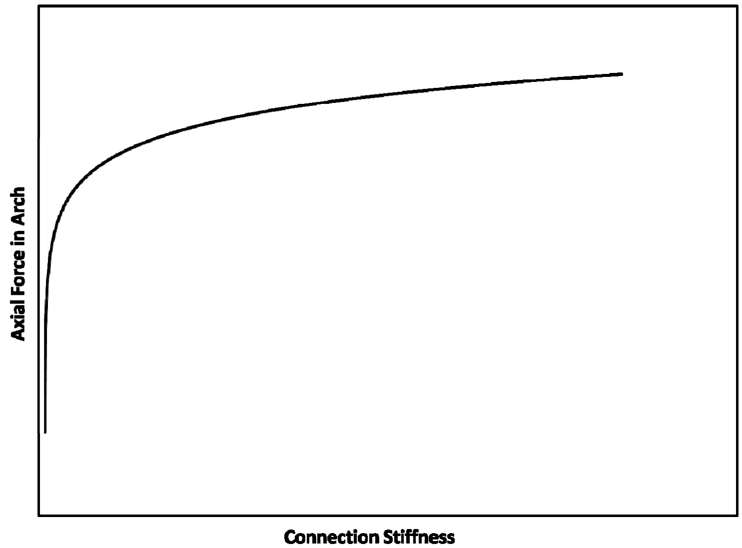


Figure 48: Qualitative relationship between connection stiffness and axial force in the arches.

9.4 Influence of Construction Sequence

Two different approaches to constructing a Burr truss were introduced in Section 2.1. The first method, used by the Kennedy family of bridge builders in Indiana, was to connect the arch to the truss prior to releasing it from its shoring. Thus, load was transferred from the truss to the arch at the start of construction. The second method, used by another Indiana builder, J. J. Daniels, did not connect the arch to the truss until the truss had some time to settle under its own weight. Thus, Daniels' method relied more heavily on the truss to carry the dead load than did the Kennedys' method.

Daniels' method, whether or not also used by Burr, demonstrates a higher level of sophistication and understanding of the Burr truss form than the Kennedys'. The multiple kingpost truss, while more stiff than a two-hinged arch under an asymmetric live load force, still requires time under load to achieve the stiffness necessary for supporting an arch subjected to concentrated live loads. The use of traditional joinery in the truss means that it is relatively flexible when first constructed. By allowing the joints to settle into place under the self-weight of the truss, Daniels ensured a stiffer "stiffening truss." The tendency to creep, or continuously displace under a sustained load, is inherent in wooden structures. As the rate of creep is highest when a structure is first loaded, Daniels' approach also allowed the most significant loss of camber to occur before the arch was connected, relieving that system of taking more than its share of the load.

Table 14 demonstrates the dead load axial forces resulting from the two methods of construction. Since Daniels' approach puts dead load into the truss members before the arch is added, the analysis conclusion seems evident. Ignoring the long-term effects of creep, Daniels' approach relies more on the kingpost truss while the Kennedy approach relies more heavily on the arches. However, wood exhibits viscoelastic behavior, that is, strain increases under constant load. Over time, as the stiffening power of the truss is compromised due to creep, the presence of the arch allows for an alternative load path to the abutments. Consistent with the results of the Barrackville Covered Bridge study, this reasoning suggests that even if Daniels' method did not rely much on the arch, over time the systems would very likely find themselves sharing the load in a manner similar to a bridge built with the Kennedy method of construction.

Table 14: Axial forces due to dead loads resulting from the different methods construction used by J. J. Daniels and the Kennedy Family

Member	Method of Construction	
	J. J. Daniels	***The Kennedy Family***
upper chord	-10624	-7462
lower chord	+5613	+1063
upper arch	-9514	-11640
lower arch	-7321	-8934
post	+6091	+4161
diagonal brace	-6522	-4665

10. CONCLUSIONS

Rehabilitation of Gilpin's Falls Covered Bridge offered engineers a unique opportunity to examine the distribution of dead load between the two systems, arch and truss, used in a Burr truss. The experimental strains measured during rehabilitation compared well with numerical structural models of the bridge, with both methods indicating that during and shortly after rehabilitation, the truss sustained a majority of the dead load on the bridge. However, it is expected that in the long-term, more weight will be transferred from the truss to the arch as the truss undergoes long-term settlement due to creep.

The short-term dominance of the truss is strongly connected to the construction sequence chosen by the bridgewright. Historically, two methods of construction have been used: either the truss is constructed first and allowed to settle under its own self weight before connecting the arch, or the arch and truss are constructed together, removing shoring only when both are complete. The former was used in the rehabilitation of Gilpin's Falls Covered Bridge, placing more weight on the truss from the start.

The arch and truss are connected with a single through-bolt at each post-to-arch intersection. The stiffness of the connection affects how much load is transferred from the truss to the arch, but unless a visible failure of the through-bolt or the arch-to-arch connection has occurred, the connection stiffness is likely not going to significantly impact the distribution of load. Additionally, for modeling purposes, as long as the connection was relatively intact, no difference was found in results obtained from the two-dimensional structural model (which ignored the out-of-plane through-bolt) and the three-dimensional model.

This study added further validation to previous researchers' claims that nineteenth-century builders likely sized the members within each system (arch and truss) to carry 100 percent of the load. However, the failure of Gilpin's Falls Covered Bridge demonstrates that even this is not enough redundancy should the ends of the arches lose their support from the abutments. In this scenario, the Burr truss becomes a tied arch wherein the arch thrust is directed into the lower chord, overburdening the lower chord splice joints. Thus, for those tasked with inspecting these bridges, it is critical that close attention be paid to the soundness of the connection between the arch and abutment.

SOURCES

Allen, Richard Sanders. *Covered Bridges of the Middle Atlantic States*. Brattleboro, Vermont: The Stephen Greene Press, 1959.

American Forest & Paper Association, Inc. (AFPA). *NDS Design Specification, Supplement: Design Values for Wood Construction*. 2005 ed. Washington, DC, 2006.

Andrews, Timothy. Conversations with Jeremy Mauro, June 2009 to July 2010.

"Baltimore and Chambersburg, Pa." *Baltimore Sun*, August 8, 1881.

Bodig, J. and B.A. Jayne. *Mechanics of Wood and Wood Composites*. Malabar, Florida: Krieger Publishing Company, 1993.

Brown, J. Edwin. "Gilpin's Covered Bridge." *Connecticut River Valley Covered Bridge Society Bulletin* 23, no. 1 (Summer 1976): 7.

Burr, Theodore. United States Letter Patent 2769X, April 3, 1817.

"Burr, Theodore," biographical sketch. In *A Biographical Dictionary of American Civil Engineers*. New York: American Society of Civil Engineers, 1972.

Cecil County Commissioners. *Records*, 1860-61.

Cecil County Historical Society. "Gilpin's Falls Covered Bridge 1860," typed manuscript, c. 1992.

Cecil Whig [Elkton, Maryland], August 18, 1860; September 8, 1860; December 15, 1860, September 18, 1936.

Conwill, Joseph D. "Burr Truss Bridge Framing." *Timber Framing* no. 78 (December 2005): 4-11.

Cooper, Theodore. "American Railroad Bridges." *Transactions of the American Society of Civil Engineers* XXI (July 1889): 1-52.

"Covered Bridges of Maryland and Delaware." *Connecticut River Valley Covered Bridge Society Bulletin* 8, no. 1 (Summer 1961): 6-7.

Cummings, H.M. "Theodore Burr and His Bridges across the Susquehanna." *Pennsylvania History* 23, no. 4 (October 1956): 476-486.

Edwards, L.N. *A Record of History and Evolution of Early American Bridges*. Orono, Maine: C.H. Edwards, 1959.

Fischetti, David. *Structural Investigations of Historic Buildings*. Hoboken, New Jersey: John Wiley & Sons, Inc., 2009.

Forest Products Laboratory. *Wood Handbook: Wood as an Engineering Material*. Madison, Wisconsin: United States Department of Agriculture Forest Service, 2010.

Gasparini, Dario A. and C. Provost. "Early Nineteenth Century Developments in Truss Design in Britain, France and the United States." *Construction History* 5 (1989): 21-33.

"Gilpin's Falls Covered Bridge." National Register of Historic Places Nomination Form, 2008.

Gould, George E. *Indiana Covered Bridges through the Years*. Indianapolis: Indiana Covered Bridge Society, Inc., 1977.

Graton, Milton S. *The Last Covered Bridge Builders*. Plymouth, New Hampshire: Clifford Nicol, 1978.

Griggs, F.E. "Standing the Test of Time: Theodore Burr's Wooden Bridge Designs." *Structure Magazine* (2004), 28-30.

Haupt, Herman. *General Theory of Bridge Construction*. New York: D. Appleton and Co., 1870.

Historic American Engineering Record (HAER), National Park Service, U.S. Department of the Interior. "Kennedy Bridge." HAER No. IN-24.

__________. "Forsythe Bridge." HAER No. IN-106.

__________. "Jackson Bridge." HAER No. IN-48.

__________. "Barrackville Bridge." HAER No. WV-8.

__________. "Pine Grove Bridge." HAER No. PA-586.

Illustrated Atlas of Cecil County, Maryland. Philadelphia: Lake, Griffing & Stevenson, 1877.

James, J.G. "The Evolution of Wooden Bridge Trusses to 1850: Part 1." *Journal of the Institute of Wood Science* 9 (1982): 116-35.

Johnston, George. *History of Cecil County, Maryland*. Elkton, Maryland, 1881.

Kemp, Emory L., ed. *American Bridge Patents: The First Century (1790-1890)*. Morgantown, West Virginia: West Virginia University Press, 2005.

Kemp, Emory L. and J. Hall. "Case Study of Burr Truss Covered Bridge." *Engineering Issues: Journal of Professional Activities* 101, no. 3 (1975): 391-412.

Kniffen, Fred. "The American Covered Bridge." *Geographical Review* 41, no. 1 (January 1951): 114-123.

Lamar, Dylan and Benjamin Schafer. "Structural Analyses of Two Historic Covered Wooden Bridges." *Journal of Bridge Engineering* 9, no. 6 (2004): 623-33.

Mauro, Jeremy. "Preserving a Nineteenth Century Bridge in the Twenty-First Century." *Conference Proceedings, Preserving the Historic Road, Washington, D.C.*, September 9-12, 2010.

Martinett, Simon J. "Map of Cecil County, Maryland, 1858."

National Society for the Preservation of Covered Bridges. *World Guide to Covered Bridges, 2009 Edition*. Concord, New Hampshire: Capital Offset Company, Inc., 2009.

O'Grady, Patrick. *The Rebuilding of the Corbin Covered Bridge in Newport, New Hampshire*. Concord, New Hampshire: Town and Country Reprographics, 1998.

Pierce, Phillip C., Robert L. Brungraber, Abba Lichtenstein, Scott Sabol, J.J. Morell, and S.T. Lebow. *Covered Bridge Manual*. Publication No. FHWA-HRT-04-098. Turner-Fairbank Highway Research Center, Federal Highway Administration, U.S. Department of Transportation, April 2005.

"Progress on Rehabilitation of Gilpin's Falls Bridge." Available at http://www.mdcoveredbridges.com/news.html, updated June 30, 2009, retrieved July 8, 2009.

Sangree, Rachel H. and Benjamin Schafer. "Field Experiments and Numerical Models for the Condition Assessment of Historic Timber Bridges: Case Study." *Journal of Bridge Engineering* 13, no. 6 (2008): 595-601.

U.S. Bureau of the Census. Federal Census for Baltimore County, Maryland, 1880, 1900, and 1910.

_________. Federal Census for Cecil County, Maryland, 1850, 1860, and 1870.