KINGS COVERED BRIDGE REHABILITATION
SOMERSET COUNTY, PENNSYLVANIA

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Fig.1: *Kings Bridge over Laurel Hill Creek after rehabilitation, east portal.*

Summary

Kings Bridge spans 116 feet over the Laurel Hill Creek. The original multiple kingpost truss structure was constructed circa 1857 by an unknown builder and retrofitted with nail-laminated arches circa 1900. Fortunately, the historic structure was bypassed with a steel bridge in the 1930s, and the original structural systems were never "modernized" like many timber bridges were to serve increased vehicular loads and highway traffic.

The total rehabilitation of Kings Covered Bridge in Somerset County, Pennsylvania was completed in 2008, culminating an eleven-year effort by the non-profit Southern Allegheny Conservancy (SAC) and a private-public partnership utilizing 100 percent federal funding from two Federal Highway Administration programs – the National Covered Bridge Program and the Transportation

Enhancements Program. The covered bridge boasts a colorful history and provides examples of rare structural systems that are now conserved, including an original lattice floor joist system under a restored deck. Most of the major historic members and much of the historic fabric were conserved in place. Arches that had been retrofitted after initial construction and tied to the lower chords of the original multiple kingpost truss system were rehabilitated. Using an innovative engineering strategy, arches were extended through the lower chords to create a hinged arch that serves like a Burr arch structure by bearing on the abutment.

The structure remains a museum-quality artefact for pedestrian use. Bridge ownership was transferred from the Southern Allegheny Conservancy (SAC,) the non-profit that managed the rehabilitation project to the local municipality after site improvements for visitors were completed in spring 2009. The Kings Bridge site now serves as one two "twin covered bridges" in municipal parks – connected by a bike route to the Barronvale Bridge located one mile upstream in Middlecreek Township.

Keywords: *Multiple Kingpost Truss, Burr Arch Truss, retrofitted arch, Somerset County, Pennsylvania, FHWA, PennDOT*

1. CHRONOLOGY of DEVELOPMENT & USE

From the 1930s until 2002, the Kings, a local farming family of Middlecreek Township, owned and maintained the bridge. The family retrofitted "Kings" Bridge to serve as a livestock barn over the water. Former gates and fences from this agricultural use were saved to re-install on the bridge after rehabilitation. Remnants of rubber tire hinges still exist on the lower downstream chord where the Kings hung a "floating fence" to prevent livestock from wandering up the creek in low flow periods.

***Fig. 2** Human and livestock gates at east portal. Note the diagonal repair rods through the truss braces, which were installed early in the twentieth*

***Fig 3**: Same gates from outside.*

Project engineers agree that it was the King Family who was responsible for ensuring the bridge's survival by maintaining the roof and installing a remarkably astute homespun system of tension rods when both lower chords began to fail. During the rehabilitation, these rods were un-tensioned and ultimately left in place to acknowledge the family's interventions and to interpret the full structural history of the bridge, in keeping with the project preservation plan.

By 1997, both lower chords had failed completely, and only the arches and the repair rods installed by the Kings prevented the bridge from imminent collapse. In 2000, a temporary support system was engineered and installed with two longitudinal, queenpost-tensioned trusses supported on timber crib towers. Transverse needle beams were installed between the two steel queenpost trusses below the upper chords to bear the suspended weight of the covered bridge until rehabilitation could be fully funded, and engineering and construction completed. The falsework remained in place through 2007 and served as the construction staging for the rehabilitation contractor, which enabled the work to be completed in place with minimal disturbance to Laurel Creek below.

The temporary stabilization system was engineered and installed, and a funding strategy was developed under an initial state-funded project for $90,000. By 2004, SAC and partners had secured $860,000 in federal funds to engineer and carry out a total rehabilitation project.

2. PHYSICAL DESCRIPTION of BRIDGE

The Kings Bridge that existed before 1997 was constructed in at least two phases. The construction of a multiple kingpost truss is estimated to date from the middle of the nineteenth century circa 1860 or earlier. The arches were added to the truss around 1900. The following description is of the extant structural system at the time of rehabilitation:

2.1 Trusses: The lengths of truss bays between posts were neither identical nor were they constructed using the typical geometric conventions of the time. Traditionally, the longer bays were near both ends, but in the Kings Bridge, the longer spans were found in the center bays where tension forces were greater in the lower chords. Several hand-hewn posts remained in the trusses, but most members were sawn – indicating later repairs. Tie beams were dropped into slots at the tops of posts and pinned with timber pins or “trunnels.” Rafter sills were bearing on the outrigger ends of the tie beams. Knee braces were joined using mortises between tie beams and posts. Horizontal X-bracing was alternately nailed and “let in” between the tie beams of each bent. Lower “needle” beams were added later to the posts, below the lower chords, suspended from the arches for supplemental support.

2.2 Arches: Nail-laminated arches, constructed of circular-sawn boards were not “let in,” but were bolted to truss members – indicating their later addition. The combined structure of trusses and arches was intended to carry the live load. Because the arches were tied to the lower chords (not hinged), they imparted a significant horizontal thrust into those members and contributed to the failure of both in locations weakened by water damage. These subsequent chord failures changed the distribution of the loads through the arches dramatically by visibly deforming them, but not to failure.

2.3 Struts: Instead of extending the arches to create a hinged arch, the early re-builders installed diagonal struts from the bottom chords (in the general location of the arch connection) to the abutment faces. These struts were similar to arch extensions in appearance but able to carry much less force. The wood struts decayed at their abutment seats and ultimately transferred no forces from the arch directly into the abutment. This resulted in greater forces imparted from the arches horizontally into the lower chords. The deteriorated lower chords failed, but the trusses tied to the arches did not fail, and instead spread each chord apart toward both ends. The abutments resisted these forces, but not without damage to the substructures that were originally built to receive only vertical loading from the multiple kingpost superstructure.

2.4 Substructure: The bridge was constructed on cut ashlar limestone abutments that had “seats” for later struts carved into their faces. As suspected, rehabilitation excavation revealed that the abutments and wingwall substructures were a single course deep, and backfilled with rubble and compacted earth.

2.5 Joists / Floor; A rare, lattice system of floor joists was used below the timber deck, where one layer of diagonal joists was overlaid by a second layer at an opposite angle. These light, circular sawn 5 x 6s were bearing on two levels of individual ledger blocks nailed to the inside face of the lower chords. Overlapping joists were not fastened at their intersections like lattice trusses. The longer joist spans created by this design appear to have combined the purposes of transverse beams, longitudinal stringer beams, and under floor diagonal bracing into one system that carried floor loads and provided lateral bracing. The lower layer of decking was laid transverse, while the upper was laid longitudinally.

Fig. 4: *Lattice floor joist system and lower tie beam added after original construction.*

2.6 Sheathing: The trusses were clad with board and batten siding. The roof consisted of deteriorated asphalt shingles installed on top of deteriorated wood shingles on circular sawn nailers that had been mounted on sawn rafters fastened at the apex with trunnels. Wainscoting and a canted cap were in place to protect the lower area of the inside of the trusses from traffic debris.

3. DESCRIPTION of STRUCTURAL ASSESSMENT / ANALYSIS

Prior to the rehabilitation project, an engineering investigation of the structural conditions of the bridge was conducted. Inspection revealed that water damage posed the greatest challenge to the bridge in three structural areas.

3.1 Lower chords: Breaches had occurred in the corresponding bays on the opposite ends of each truss. These weaknesses (breaks or failures) were recognized in time by the Kings, who skillfully made a series of vernacular repairs, including metal rods, wood splints and iron brackets to keep the bridge standing. Removing wainscoting during the engineering investigation revealed that wood scabs had been added across the faces of truss posts and braces in the area of failure. Several areas of deterioration occurred in those sections of the lower chords fastened by traditional “lightning bolt” tension splices. This complicated the repairs by requiring removal of the entire length of a chord section or splicing in short new chord sections.

3.2 Truss Posts: Roof leaks above the lower chord failures also resulted in damage to joints and several truss posts and shouldered truss braces in Kings Bridge. The heads of two posts failed in shear at the joints due to excessive new loading patterns resulting from the lower chord failures. Several of the posts were found to be only partially damaged, while one required total replacement.

3.3 Arches / Struts: The arches were visibly deformed above the locations of lower chord failures, exhibiting the transfer of loads from the adjacent truss posts through the arch and back into the trusses and lower chords to the foundations. Struts were installed below the lower chords and were never sheathed by the bridge siding. The hearts of these untreated heavy timbers decayed at the bearing seats and remained in place suspended by metal tie rods only

3.4 Non-destructive testing: In 2004, the rehabilitation design team inspected the structure, assisted by personnel from the USDA Forest Service and the Forest Products Laboratory (FPL) in Madison, Wisconsin. An FPL engineer performed a series of non-destructive evaluation tests on various bridge truss members that were identified as deteriorated or potentially deteriorated based upon a visual condition assessment. The non-destructive testing (NDT) techniques included moisture content testing, stress wave analysis, and resistance drilling. Samples were retrieved from the structure in locations where member replacement was imminent, and those were also tested by FPL for species and strength. It was determined that white oak was the original species to be matched where replacement of structural members was required. All adjusted moisture content measurements were less than 16 percent, except for one test location that was slightly higher at 19 percent. These results indicated that the truss members were drier than the threshold moisture content level required for decay. Stress wave velocities ranged from 180-220 ft/μ-second and were near the threshold level for the presence of internal deterioration. Several micro-drill resistance measurements reported consistently low wood density and confirmed the stress wave measurements. Both truss members had a relative drilling resistance below 15 percent, with the interface between members visible at approximately 7.5 inches drilling depth.

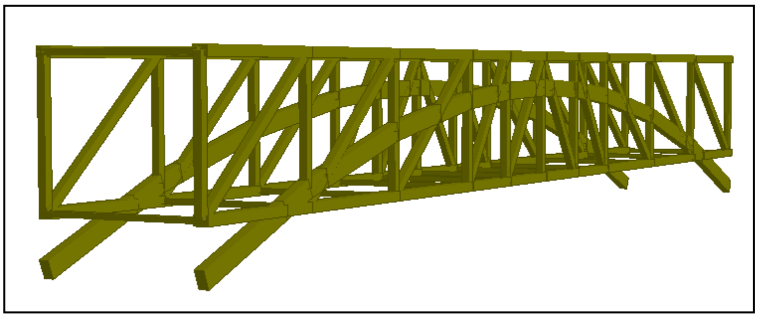
3.3 Modeling: The arch-truss system of Kings Bridge was modeled and analysed, assuming a linear-elastic behaviour using STAAD structural analysis software. The geometry of the bridge was developed based upon centerlines of the members measured directly from the bridge in its existing state. Section and material properties were also used to describe the members. The material properties of the white oak truss members were determined from small scale laboratory testing conducted by the Forest Products Laboratory. The tests conducted by FPL in October 2004 also included various non-destructive techniques conducted on site on selected bridge members to determine moisture content, decay, and defects (i.e., checks, splits, etc). In addition, mechanical testing conducted by FPL on small-scale samples retrieved from the bridge determined species, specific gravity, moisture content, MOE, and MOR values. An MOE value of 1.4 x 106 lb/in2 was used in the analysis.

Due to the complexity of the geometry and variety of connections, a conservative approach was followed in the modeling. A primary issue was the behaviour of the joints at the intersections of verticals, diagonals, chord members, and the arches. The ends of the diagonals and the ends of the posts were assumed to be pinned (i.e., free to rotate). The chord members and the arches were assumed continuous, and all other joints were assumed fixed. This was considered to be a conservative approach and an accurate model of the bridge, and is in line with earlier analyses of Burr arch-truss systems. The supports of the truss were modeled as pinned at the left end and roller supported (resisting only vertical movement) at the other end. This was considered to be the most probable state of the original construction and to have generated the greatest forces in the bottom chord. The arches were modeled as pinned at both ends and resting in a corner of the stone abutment. The arch, which actually is a double arch (i.e., each half of the arch is composed of nine nail-laminated 2" x 4" members) straddling the truss members, was approximated by a series of twenty straight members and was modeled as a continuous member.

Dead loads were approximated by measuring timber dimensions on site, including truss members, top and bottom bracing, deck, roofing, siding, etc. These volumes were multiplied by a unit weight of white oak approximated at 43 lb/ft3 and placed at upper and lower chord panel joints in a manner approximating the actual loading conditions. Physical testing of small samples retrieved from Kings Bridge indicated that the specific gravities ranged from 0.59 to 0.68, which corresponded well with published values of density in the Wood Handbook. The live load used was a pedestrian load of 85 lb/ft2 as specified by AASHTO Guide Specifications for Design of Pedestrian Bridges published in 1997. The live load was divided between the two trusses and placed at the lower chord panel joints in a manner approximating the actual loading conditions. Since the AASHTO bridge specifications and PennDOT DM4 Bridge Design Manual do not adequately address the issue of wind load or snow loads for covered bridges, a conservative approach was followed in selecting snow and wind loads and load combinations. A snow load of 35 lb/ft2 was used and placed at the upper chord panel joints. A wind load pressure of 12.5 lb/ft2 based on ANSI/ASCE 7, Minimum Design Loads for Buildings and Other Structures, Section 6.4, Method I Simplified Procedure, corresponding to a basic wind velocity of 100 mph was used and divided between the upper and lower chord panel joints in a manner approximating the actual loading conditions.

In this analysis, a combination of dead, live, wind, and snow loads was followed. The load combination used was in accordance with ANSI/ASCE 7, Minimum Design Loads for Buildings and Other Structures (Section 2.4), Dead + (Wind + Live + Snow)*0.75, which is considered conservative for analysing extant covered bridges (according to the Federal Highway Administration's *Covered Bridge Manual*) A three dimensional (3-D) space frame model of the bridge was developed using STAAD with the loads applied at the top and bottom panel joints as per the ANSI/ASCE 7 load combinations. The versatility of the 3-D model made possible the addition of the top X-bracing members and their analysis for the lateral loads. The 3-D model also made possible a very realistic representation of the actual structure. The continuity of the top and bottom chords and the arches was preserved, although the bottom chord was severely damaged (i.e., ruptured) at two locations that were not modeled.

Figure 5: 3-D Rendering (STAAD Model) of Kings Bridge



The results from the STAAD analysis containing member forces, moments, and joint deflections were analysed with two objectives in mind: (1) to compare the computed stresses with allowable stresses (obtained from the National Design Specification (NDS) for wood construction) to ascertain the degree of safety of the member in question; and (2) to achieve some understanding of how the arch-truss system behaves, i.e., stress distribution and deflections. From the analysis it was determined that the most important structural characteristic of the arch-truss system of the Kings Covered Bridge when compared to the original multiple kingpost system was the stiffness associated with deflections. The addition of the Burr arch greatly reduced the deflection by a factor of approximately three under full load, suggesting the synergy of both the arch and truss producing a structure that is stiffer than the kingpost truss acting alone. For long span timber bridges, such as Kings Bridge, the arch provides a necessary stiffening of the truss so that deflections resulting from live and dead loads are controlled to acceptable limits. The addition of the arch also accomplished a reduction of member stresses and even the critical lower chord-arch joint member was satisfactory.

4. TREATMENTS / INTERVENTIONS

Rehabilitation methods and techniques were conceived and conducted in accordance with the Secretary of the Interior's Standards for the Rehabilitation of Historic Properties. The rehabilitation of Kings Bridge minimized interventions and required repairs in-place without dismantling and replacement of deteriorated members in-kind where possible.

The engineering strategy adapted conserved the remaining structural integrity of Kings Covered Bridge and focused on rehabilitating and stiffening the trusses by extending the arches to bear directly on the stone abutments while employing traditional nineteenth-century timber joinery methods such as "joggle" splice joints to replace deteriorated members. Epoxy adhesives and Glass Fiber Reinforced Polymer (GFRP) rebars and plates were also used to splice new members to existing ones, as well as to stiffen members. Only where rot damage was so severe that repairs could not be made in compliance with conservation best practices were irretrievable members replaced with other wood material. The first option for replacement timber was from salvaged members of the bridge that had been removed because they were deemed to be not historically significant and were not designated for repair.   The rehabilitation project focused on the following areas:

4.1 Extend arches: The arches were extended below the lower chords by adding in-kind laminations to the existing nail-laminated arches. The cross-section area of the arch was removed from the corresponding section of each lower chord member to allow the new arch extension to bypass the chord unfastened.

Fig. 6*: Arch "tied" to chord and strut below to abutment.*

Fig 7: *Laminated arch extended to reinforced abutment.*

While the bridge was elevated for rehabilitation, the new arch extensions were installed and scribed to meet the new seats carved in the stone abutments before the bridge was lowered into place. The ends of the timbers were treated with wood preservative and elastomeric pads were used as compressive and thermal insulators between the stone and wood at the bearings. The new arch extensions were intended to remedy the damage to the trusses caused by the previous tied arch geometry. The deformed arch sections were separated, reshaped, and re-laminated in place. Repairs were also made by extending the laminations past the lower chords by dapping (notching) a portion of the bottom chord members. The new laminations follow the existing butt joint pattern and are also nail-laminated, stitched, and joined with epoxy. This repair includes the addition of a GFRP flitch plate with epoxy applied to both sides of the GFRP plate to reinforce the notched bottom chord.

4.2 Rebuild / reinforce stone abutments: A system of abutment reinforcements was designed to buttress the existing stone abutments. The abutment / wingwalls were completely excavated and formed to receive flowable backfill to create the buttress that would withstand the new forces created by the extended arches. An estimated 20 percent of the substructure stone was removed and re-laid in the rehabilitation process. All former Portland cement grouting was removed, and all joints were re-struck with a softer mix using a greater percentage of lime to avoid spalling at the edges of the softer stone with the harder mortar. New white oak sills were installed to replace the previous sills that had deformed under the horizontal loads imparted after the chord breaches.

Fig 8*: Excavated abutment before reinforcement.*

Replace posts, braces and chords (full and partial): Based on visual inspection, the non-destructive testing conducted by FPL, and the results of the structural analysis, several repair types were implemented. Glass Fiber Reinforced Polymer (GFRP) reinforcing rods embedded in epoxy were used to repair existing timber members where deterioration was such that an epoxy mortar repair was not appropriate, where a new section of wood was to be added to an existing original member, or where a fractured original member was restored. Another repair type intended to repair an existing fractured or missing timber section required the installation of a new white oak implant "joggle" at the splice location. This repair method represents a traditional timber joinery splice consistent with that period.

Fig.9*: Joggle splice repair (above).*

Fig. 10*: GFRP post repair (opposite).*

Restore other members and sheathing materials: The members of the lattice joist floor system were replaced in-kind. Some of the decking was reused as subflooring, but most of the remainder was replaced by rough-sawn hemlock boards. Both courses were laid transverse with staggered joints. Some rafters and all lath were replaced in-kind.

Figure 11*: Interior rehabilitated.*

Rough-hewn cedar shingles were installed to the lath. Most of the siding was replaced in-kind, with some reused as battens. The wainscoting and cap material was stockpiled and then reinstalled.

Figure 12*: South elevation – rehabilitated.*

5. CONCLUSION

The abutments remained in place and a system of abutment reinforcements were installed to buttress the existing stonework. Archaeological al disturbance did not take place. The laminated arches were extended to the abutments and the bridge continues to operate as a self-supporting wood truss without modern support. The historic resource now serves as a pedestrian crossing of the Laurel Creek and is one half of a new “twin covered bridge” municipal park connected by a trail to the Barronvale Bridge one mile upstream, also owned by Middlecreek Township.

The visual character of the bridge was not compromised and all character-defining features were retained. The geometry of the bridge, such as the nail-laminated arches, was altered with minimal interventions to remedy fatal structural defects. The changes, such as the extension of the nail-laminated arches, were deemed structurally necessary to remedy the damage to the trusses caused by the previous arch installation. Every member of the truss was carefully examined and retained if possible. Any new materials match in-kind the materials they replaced or were based on historical antecedent. No changes were made that would create a false sense of historical development. All character-defining features were respected as products of their time within the period of significance. Glass Fiber Reinforced Polymer (GFRP) reinforcing rods were also used as a method of repairing existing timber members (*as opposed to total replacement*) where deterioration was found to such extent that an epoxy mortar repair was not appropriate and where a new section of wood was to be added to an existing original member.

The most important members of the truss were preserved. Replacement members were made from in-kind material and traditional craftsmanship including joinery techniques that replicated historic construction methods. Repair work was completed with in-kind materials and traditional craftsmanship techniques including joinery methods. Other non-traditional repair was carried out using GFRP rods to repair and strengthen existing members.

6. REFERENCES

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7. REHABILIATION PROJECT TEAM

Rehabilitation Design:	Simone Collins Landscape Architecture
Structural Engineer:	Gannett Fleming Inc.
Stabilization Engineer:	DCF Engineering Inc.
Stabilization Contractor:	Arnold M. Graton Associates
Construction Contractor:	Allegheny Restoration
Technical Assistance:	USDA Forest Service
	Forest Products Laboratory
	Somerset County Conservation Service
Funding:	Federal Highway Administration
	PA Department of Conservation and Natural Resources
	PA Department of Community and Economic Development
	Somerset County
Project Partners:	Pennsylvania Department of Transportation
	Somerset County
	Middlecreek Township
	Rockwood Area Historical Society
Owner:	Middlecreek Township