Structural Analyses of Two Historic Covered Wooden Bridges

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Abstract: Structural analyses were conducted of two covered wooden bridges still in use in the Summer of 2002. The Pine Grove Bridge and Brown Bridge represent the 19th century truss forms of the Burr-arch truss and the Town-lattice truss, respectively. In the Burr-arch truss, the arch is shown to be dominant in carrying the dead load of the bridge, while the truss, following simple beam behavior, primarily provides resistance against concentrated live loads. The use of camber and a common retrofit of steel ties are found to have varied effects. The Town-lattice truss is found to follow simple beam behavior with stress concentrations at the supports. Studies are conducted to highlight the sizing and placement of the chords, the behavior and advantages of bolster beams, and the sizing and placement of the lattice members. It is hoped that this study may aid those wishing to better understand these bridges' historic significance and demonstrate how modern engineering analysis may aid our understanding of historic bridges.

DOI: 10.1061/(ASCE)1084-0702(2004)9:6(623)

CE Database subject headings: Structural analysis; Bridges, wooden; Bridges, covered; Historic sites; Pennsylvania, Vermont.

Introduction

The quaint beauty of our nation's covered wooden bridges has seldom gone unnoticed. However, beyond their aesthetic beauty, in these structures we can also find beauty in their engineering prowess. By studying the timber truss designs of the early 19th century, we can gain a greater understanding of the rich engineering heritage of the United States. This study focuses specifically on two covered wooden bridges typical of the 19th century. The first, Pine Grove Bridge (Fig. 1), is an 1884 Burr-arch–truss structure located on the Chester and Lancaster county line in southeastern Pennsylvania. The second, Brown Bridge (Fig. 2), was constructed in 1880 of the Town-lattice truss, and is located in Rutland County in south central Vermont. These bridges were studied by the writers as part of a larger National Park Service program through the Historic American Engineering Record (HAER) that is examining and documenting significant covered wooden bridges in the United States (HAER 2003a,b).

The development of wooden-truss bridges in the United States began in the late 18th century. The first design to achieve widespread popularity was the arch-truss patented by Theodore Burr in 1817. Soon after, in 1820, Ithiel Town patented his lattice truss. Truss development continued with the patents of Long and Howe in 1830 and 1840, respectively; however, both the Burr- and the Town-truss forms gained widespread acceptance and were fully utilized until the abatement of timber bridges in the early 20th century (James 1982).

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Note. Discussion open until April 1, 2005. Separate discussions must be submitted for individual papers. To extend the closing date by one month, a written request must be filed with the ASCE Managing Editor. The manuscript for this paper was submitted for review and possible publication on January 23, 2003; approved on June 24, 2003. This paper is part of the ***Journal of Bridge Engineering***, Vol. 9, No. 6, November 1, 2004. ©ASCE, ISSN 1084-0702/2004/6-623–633/$18.00.

Although the specifics of the original design of the Pine Grove and Brown bridges are not known, both Elias McMellen and Nichols Powers, the builders of the Pine Grove and Brown bridges, respectively, were experienced timber bridge builders. Further historical details can be found in the HAER documents (2003a,b). The analysis of the Pine Grove Bridge contained herein demonstrates the dominance of the arch, but necessity of the truss, in the Burr-arch–truss design. Further, the role of camber and retrofits, such as steel tie rods, are specifically examined. Work on the Brown Bridge shows generally anticipated behavior, but also subtleties in chord sizing that suggest the builder's (Powers) significant understanding of the bridge's behavior. Further, explicit examinations of features of the Town lattice demonstrate the importance of the bolster beams and explore the necessity of the secondary chords and lattice members.

Structural Analyses

To examine the structural behavior of these bridges, planar models of the trusses are generated and analyzed assuming linear-elastic behavior (McGuire et al. 1999). The bridge dimensions are based upon centerlines of the members measured directly from the bridges in the Summer of 2002 [see HAER documents for dimensions (2003a,b)]. As modeled, the clear span of the Burr-arch truss is 27.4 m (90 ft) and the Town-lattice-truss is 31.1 m (102 ft).

For the Burr-arch truss, the modulus of elasticity is estimated at 8,274 MPa (1,200 ksi) based on the suspected wood species of Eastern hemlock or Eastern white pine. For the Town-lattice truss, the modulus of elasticity is estimated at 9,653 MPa (1,400 ksi) based on the suspected wood species of Eastern spruce [FPL 1999]. Dead loads have been approximated by measuring lumber dimensions on site, including truss members, roofing, siding, etc. These volumes have then been multiplied by a unit weight of 561 $\mathrm{kg/m^3}$(35 pcf) and the loads placed at joints in a manner that approximates the actual loading distribution. Since wood is a highly variable material, selections for elasticity and unit weight

Fig. 1. Photograph of Pine Grove Bridge

values are only approximate. Exact treatment of the random nature of these variables is desirable, but beyond the scope of this work. The structural analyses presented herein do not attempt to model connection specifics beyond considering them appropriately pinned or fixed. The local details of timber connections, whether it be trunnels, scarf joints, etc., are not treated in detail—thus, the analysis presented reflects the overall behavior and focuses on member, not connection, performance. However, the writers believe that the basic behavior of these trusses and the magnitudes of the stresses found are accurate for purposes of understanding and appreciating their structural behavior.

Burr-Arch Truss

The model and labeling system of the Burr-arch truss is shown in Fig. 3. The supports of the truss are modeled as pinned at the left end and roller supported at the right end. The arch is pinned at

Fig. 2. Photograph of Brown Bridge

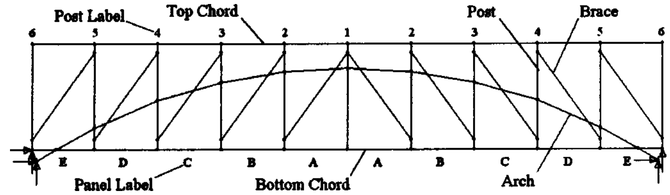

Fig. 3. Centerline model and labeling system of the Burr-arch truss

both ends. Although the Pine Grove Bridge is a two-span bridge, we have assumed each span is independent of the other due to specifics of the construction details; thus only one typical span is modeled. While an earlier analysis of a typical Burr-arch truss carried the braces to the corners where the posts and chords meet (Kemp and Hall 1975), the Pine Grove Bridge has a substantial gap between these joints. Therefore, the braces were terminated at intersections within the posts before reaching the chords (though the post is modeled as continuous through this point). This modeling detail more accurately reflects the local moment and shear demands placed on the truss verticals.

Two models of the connections in the Burr-arch truss were developed: Flexible and rigid. The flexible model assumes pin connections at the ends of the braces and the ends of the posts. The chords and arch are continuous and all other joints are assumed fixed. The rigid connection model assumes all joints to be perfectly fixed. Actual bridge behavior is expected to be closer to the flexible connection model than the rigid model.

Live loading of the Burr-arch truss is modeled as a 44 kN (10 k) concentrated load divided between the two trusses. The maximum weight limit of covered wooden bridges in Lancaster County, Pa., 44 kN (10 k), where the selected truss is located. The load is first applied at midspan of the lower chord and then at the approximate quarter point of the truss (two panel points from the end). To examine the structural behavior of the Burr-arch truss, the truss and arch components are first analyzed separately, followed by an analysis of the combined system.

Burr-Arch Truss: Truss-Only Model

A common manner of conceptualizing the "global" structural behavior of a truss is to imagine an analogous beam. The chords of a truss typically mirror the moment distribution of the beam, and the braces and posts typically follow the shear distribution.

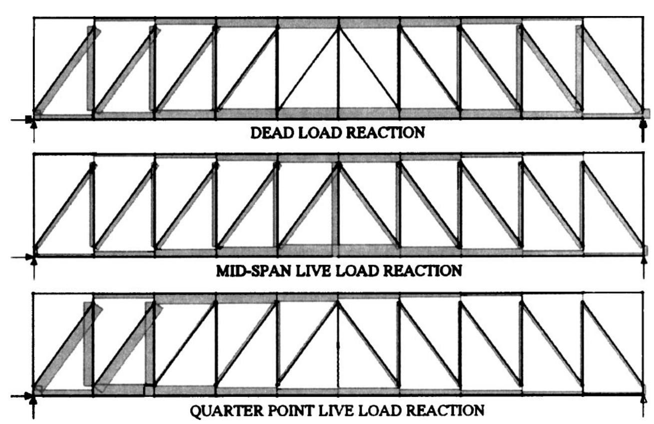

Fig. 4. Axial forces of the truss of the Burr-arch truss due to three loading conditions

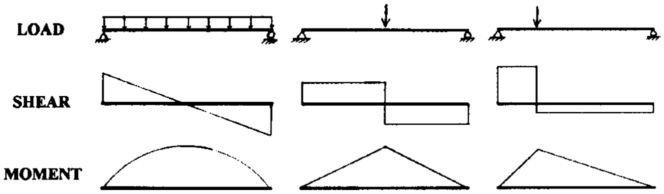

Fig. 5. Shear and moment diagrams of a simple beam under three loading conditions

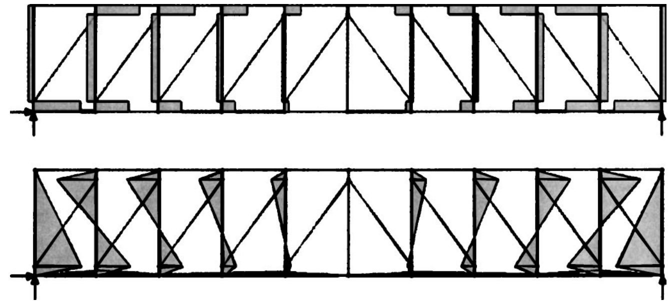

Fig. 6. Shear (top) and moment (bottom) of the truss due to dead load

Shown in Fig. 4 are the axial force diagrams of the truss components of the Burr-arch truss under three loading conditions: Dead load, midspan live load, and quarter point live load. Following this, in Fig. 5, the shear and moment diagrams of a simple beam under similar loading conditions are provided. As seen in Fig. 4, under uniform dead load, the top chord acts in compression (graphs below the element correspond to compression), and the bottom chord acts in tension (graphs above the element correspond to tension). The braces are in compression, and the posts are in tension (for vertical elements, left is tension, right is compression). Comparing with Fig. 5, chord forces of the truss represent the global moment of the truss, and are greatest where the beam's moment is greatest. The brace and post forces of the truss represent the global shear of the truss, and are proportional to the beam's shear diagrams. Note, however, that the last post (above the diagonal) and the end of the top chord have no axial force; thus they do not exhibit the global beam behavior. At this point, the reader is explicitly reminded that the global shear demand generated from the beam analogy (Fig. 5) should not be confused with the local member shear force of Fig. 6.

Table 1 contains the maximum stress values and deflections for the truss under the analyzed loading conditions. The stresses listed in this and all following tables consider the extreme fiber of the member and include the effects of moment. For comparison, the strength properties of the suspected wood species of the truss are provided in Table 2. The National Design Specification (NDS) values are design values while the values of the Forest Products Laboratory (FPL) are based on an average of test results without adjustments (AFPA 1997; FPL 1999).

By comparing Tables 1 and 2, it is seen that under dead load only, the truss is overstressed by today's design standards. Thus, it is possible that the truss alone, without the arch, would not have been a sufficient design. However, the maximum deflection value listed, equivalent to $L/900$, is acceptable considering a deflection limit of $L/300$ for highway bridge stringers [AITC 1994]. Such deflection comparisons are meant only to give a sense of the relative flexibility and in no way represent specific, applicable, design criteria.

The local shear and moments of the truss elements under dead load are displayed in Fig. 6. The posts are the only members containing significant shear and moment since they receive transverse loading by the braces. The maximum shear stress, 807 kPa (117 psi), occurring at the bottom of the end posts is in excess of allowable design values. The fact that the connection of these two elements is accomplished by notching of the post makes this shear stress even more critical (although the notched section was not accounted for in the determination of shear stress).

Since the joints of the end posts and the braces are over-

Table 1. Maximum Stresses and Deflections of the Truss

Parameters	Dead load	Midspan live load	Quarter point live load	Dead + midspan live load	Dead + quarter point live load
Maximum compressive stress [kPa(psi)]	−8,322 (−1207)	−1,048 (−152)	1,703 (−247)	−9,370 (−1,359)	−10,030 (−1,455)
Location	Post 6[a]	Post 6	Post 6	Post 6	Post 6
Maximum tensile stress [kPa (psi)]	7,157 (1,038)	986 (143)	683 (99)	8,053 (1,168)	8,625 (1,251)
Location	Post 5	Post 5	Post 4	Post 5	Post 5
Maximum deflection [cm (in.)]	2.44 (0.96)	0.48 (0.19)	0.33 (0.13)	2.95 (1.16)	2.69 (1.06)
Location	Midspan	Midspan	Panel point 4	Midspan	Midspan

Note: Stresses consider the extreme fiber of the member and include the effects of moment.

[a]Stresses occur only below intersection of last diagonal to Post 6 (see Fig. 4).

Table 2. Maximum Strengths of Suspect Wood Species

	NDS maximum allowable stress[a]			FPL maximum strength[b]	
Wood species	Compression, ∥ [kPa (psi)]	Shear, ∥ [kPa (psi)]	Tension, ∥ [kPa (psi)]	Compression, ∥ [kPa (psi)]	Shear, ∥ [kPa (psi)]
Eastern hemlock [kPa (psi)]	−6,900 (−1,000)	550 (80)	6,380 (925)	−37,300 (−5,410)	7,310 (1,060)
Eastern white pine [kPa (psi)]	−5,000 (−725)	450 (65)	4,830 (700)	−33,100 (−4,800)	6,210 (900)

Note: "∥"=Property strength parallel to the wood grain (shear strength parallel to grain is the limiting strength even when loaded transversely). NDS=National Design Specification; and FPL = Forest Products Laboratory.

[a]Values shown do not contain adjustment factors for safety or resistance.

[b]Values for tension parallel to grain are not available for large timbers.

Table 3. Maximim Stresses and Deflections of the Arch

Parameter	Dead load	Midspan live load	Quarter point live load	Dead + midspan live load	Dead + quarter point live load
Maximum CP [kPa (psi)]	−4,020 (−583)	−8,612 (−1,249)	−12,330 (−1,788)	−12,630 (−1,832)	−12,790 (−1,855)
Location	Midspan	Midspan	Panel D, side of loading	Midspan	Panel D, side of loading
Maximum deflection [cm(in.)]	2.3 (0.91)	5.1 (2.01)	12.1 (4.76)	7.4 (2.92)	12.0 (4.73)
Location	Midspan	Midspan	Panel point 4	Midspan	Panel point 4

Note: CP=compressive stress.

stressed by the current design code for both axial and shear stress, the truss form without the arch reinforcement would not be adequate in the case of the Pine Grove Bridge by current standards.

The rigid model of the truss (all connections fixed) was analyzed under dead load and it was found that although several stresses increase, the maximum stresses are lower than those of the flexible model. It is also noted that the midspan deflection is 30% larger in the pinned model than in the fixed.

Burr-Arch Truss: Arch-Only Model

With an understanding of the truss behavior, let us now consider the arch as a separate structural element. The arch is loaded with the full dead load, of the entire bridge (including the truss) to more easily compare it to the truss-only structure. The results of analyzing the arch under the same loading conditions as the truss are shown in Table 3. Under dead load, the arch has approximately the same stiffness as the truss, but, as a funicular shape for dead load, it carries the load far more efficiently than the truss. The maximum stress in the arch alone is less than half that of the truss alone. The result of applying a 22.2 kN (5 k) load at midspan, without dead load, reveals the weakness of an arch for concentrated loads. Stresses and deflection are over twice that of the dead load alone, and also in excess of maximum design stresses. An elastic buckling analysis revealed that in-plane buckling was not a problem. The combined effect of dead and midspan live load yields the maximum deflection, which is about 250% greater than the deflection of the truss under this loading. Also, the maximum stress is about 30% greater than the maximum stress of the truss for this loading. This is telling of the weakness of an arch in carrying concentrated loads. Quarter point live loading of the arch induces large moments and large deformations, and for this reason tends to be the worst case of loading for an arch. The largest deflection under dead and quarter point live load is almost 4.5 times that of the truss system.

Burr-Arch Truss: Combined Model

The most intriguing aspect of the combined arch–truss behavior is the deflection of the combined structure as compared to that of its component parts. The truss alone deflected 2.4 cm (0.96 in.) under dead load, and the arch 2.3 cm (0.91 in.); but the combined system deflects only 0.6 cm (0.25 in.). This equates to a stiffness of the combined arch truss which is nearly two times greater than a simple parallel combination of the arch and truss stiffness. The arch truss system is more than an addition of an arch to a truss or a truss to an arch, but a synergy of the two.

The axial force diagram of the arch truss under the three loading conditions is shown in Fig. 7. Under dead load, the arch members carry significantly greater forces than do the truss members (the largest arch force is 350% greater than the largest truss force). Compared with the individual arch and truss models, the maximum stress in the arch decreases by 33% while the maximum truss stress decreases by 77%. Further, the vertical force at the supports is twice as large for the arch than for the truss. It is clear that the arch is structurally dominant under dead load.

Midspan live loading absent of dead load provides insight into the arch–truss interaction. In the arch-only case for this loading, the moment at midspan is 34,300 N m (25,300 lb ft), but decreases to 1,100 N m (830 lb ft) when combined with the truss. Thus, it seems the chords of the truss are dominant in carrying global moment here.

The axial force diagram for midspan loading shown in Fig. 7 reveals large forces at midspan in the top *and* bottom chords, not seen in the dead load case. While it appears that the forces here form a couple which resist the global moment, this is an inadequate explanation since dead load (which produces a greater global moment at midspan) does not elicit the large lower chord force. The difference between uniform loading (dead load) and midspan loading lies in the shear distribution. Under midspan loading, significant shear occurs at midspan which is carried by the braces. The braces then induce tension in the lower chord. This does not occur under dead load since the shear is then negligible at midspan.

The bottom diagram of Fig. 7 displays the axial forces due to quarter point live load (plus and minus signs have been added for clarity). Of note here are the significant tensile forces which arise in the braces just to the right of the loading and decrease toward midspan. This presents a problem since the brace/post connection is only designed for bearing in compression. It was found that, when combining dead load with this quarter point live load, the brace of the left-of-center panel retains a small tensile force of 670 N (150 lb). In the actual structure, the brace simply butts into

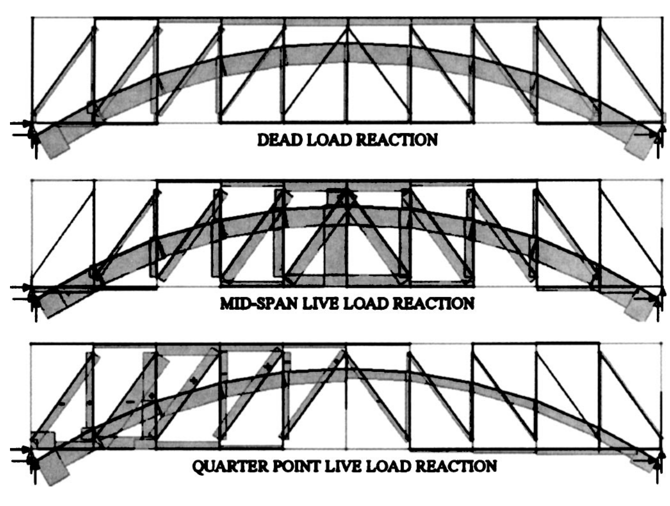

Fig. 7. Axial forces of the arch-truss due to three loading conditions

a notch in the post and has then been "toe nailed" with a sizable spike to hold it in place. The toe nail would probably not provide a sufficient transmission of this tension over the life of the structure, so some movement of this brace over time would probably be evident if tension did occur regularly. It has been suggested (not without controversy) that bridges built with "substantial initial camber" tend not to have these loading reversal problems in the braces (Pierce 1999). The Pine Grove Bridge does have a small camber, not included in this model, which is addressed in the following section.

The maximum stresses and deflections of the arch truss under the prescribed loadings are provided in Table 4. Dead load reaction dominates since the dead load is about ten times the live load. The combined dead and midspan live load case elicits the greatest deflection, a mere 0.8 cm (0.32 in.), or an impressively small, $L/3{,}300$. The combined dead and quarter point live load case produces the greatest stresses of any case considered, occurring at the left end of the arch at 3,370 kPa (489 psi). The axial force at this location is 375% greater than the largest force in the truss, again speaking for the arch's structural dominance. This maximum stress is well below current design values, suggesting that the members are sized in a conservative nature. Additionally, the low stresses on the members suggest that serviceability issues (deflections and vibrations) may have played a larger role in the actual member sizing than strength. However, as stated previously the connections were not investigated in detail, and clearly have some bearing on the preceding conclusion.

Fig. 8 displays the local shear and bending moment diagrams of the arch–truss elements under dead load. The trend for the truss is approximately the same with or without the arch. The largest magnitudes of both shear and moment are found at the ends of the span, where the global shear is greatest. The greatest shear stress of the analyzed loadings is safely below allowable limits.

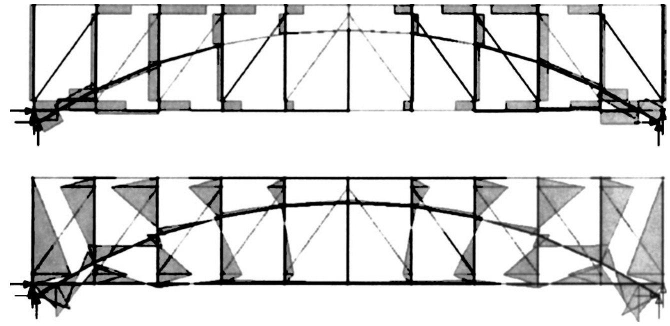

Fig. 8. Shear (top) and moment (bottom) of arch-truss due to dead load

Analysis of the rigid model of the arch truss reveals that the behavior does not drastically change from the flexible model behavior. Larger moments occur which increase some stresses slightly, but overall stresses remain well below the maximum allowable range. It is expected that the actual arch–truss behavior is safely bounded between these two limiting conditions.

Considering Effects of Camber in the Burr-Arch Truss

In the Pine Grove Bridge, there is currently (Summer 2002) a 23 cm (9 in.) camber at midspan, although it was probably greater when first constructed, before the effects of creep and joint loosening occurred. While the previous models neglected this camber

Table 4. Maximum Stresses and Deflections of the Arch Tuss

Parameter	Dead load	Midspan live load	Quarter point live load	Dead + midspan live load	Dead + quarter point live load
Arch maximum CP [kPa (psi)]	−2,700 (−391)	−470 (−68)	−680 (−98)	−2,960 (−429)	−3,370 (−489)
Location	Ends	Ends	Ends	Ends	Left end
Truss maximum CP [kPa(psi)]	−1,880 (−272)	−400(−58)	−460(−67)	−2,100 (−305)	−2,720 (−394)
Location	Post 4	Top chord	Panel D brace	Post 4	Post 4
Maximum tensile stress [kPa (psi)]	1,800 (261)	820 (119)	841 (122)	2,040 (296)	3,160 (458)
Location	Post 5	Post 2	Post 4	Post 5	Post 4
Maximum deflection [cm (in.)]	0.64 (0.25)	0.18 (0.07)	0.15 (0.06)	0.81 (0.32)	0.69 (0.27)
Location	Midspan	Midspan	Post 4	Midspan	Post 2

Note: CP=compressive stress.

Table 5. Minimum Axial Stresses due to Dead Load With and Without Camber

Stress	Without camber	With camber	Change (%)
Upper chord [kPa (psi)]	−1,150(−167)	−1,100 (−160)	−4
Lower chord [kPa (psi)]	−1,270 (−184)	−1,390 (−202)	10
Brace [kPa (psi)]	−979 (−142)	−965 (−140)	−1
Post [kPa (psi)]	−1,880 (−272)	−1,830 (−266)	−2
Arch [kPa (psi)]	−2,700 (−391)	−2,870 (−416)	6
Deflection [cm (in.)]	0.63 (0.25)	0.58 (0.23)	−8

and assumed perfectly horizontal chords, the effects of its presence on stress distribution should be understood. (It is assumed that the camber itself generates no prestressing in the members—i.e., the bridge is built in the cambered state.) An analysis of this system under dead load resulted in comparable values to the previous horizontal-chord model, as seen in Table 5. Note that the deflection was measured from the original chord position of each model. With camber, stresses are generally decreased in the truss members and increased in the arch. The primary exception to this is the lower chord of the truss which sees an increase in stress in the compressive segment, due to the lower chord beginning to act as a shallow arch. The original camber has successfully counteracted long-term deflections so that the bridge does not sag, and the stresses of the truss members, aside from the compression in the lower chord, are reduced. However, the arch and lower chord undergo greater compressive forces due to the camber, thus increasing the greatest stress of the bridge (at the arch ends).

Regarding the 670 N (150 lb) tensile force of the Panel A brace, observed under dead and quarter point live load, the presence of camber actually *increased* this force to 2,670 N (600 lb). It seems the toe-nailed spike or the connections of the braces themselves must withstand some tension. At one location of the bridge, the midspan intersection of the braces is patched by the addition of a metal cover plate over the brace to postconnection. The plate provides evidence that problems may have existed with these braces in the past.

From a serviceability standpoint, the camber is effective, but some tradeoff in strength must be made. If the first failure mode is in the crushing of the arch ends or strength of the abutment at the arch support, then the camber is detrimental. Also, camber may have detrimental effects on brace compression connections for various loading conditions.

Considering Effects of the Steel Tie Retrofit

Another complication of Burr-arch trusses is found in the addition of steel ties between the arch and lower chord as seen in Fig. 9. The steel ties provide a redundant load path in the event of failure of the post/lower chord connection. Analysis of the Pine Grove Bridge in an undamaged condition with these ties under dead and midspan live load revealed that the ties receive little force compared to the posts. Indeed, during fieldwork in the Summer of 2002, it was observed that many of the ties were quite loose. However, considering the bridge with a single ruptured post/lower chord connection at Post 3 (Fig. 10), shows that the tie carries most of the post's former load, and the forces of Panel B are largely redistributed. While the effect of the ties appears negligible in an undamaged truss, it is beneficial for increasing redundancy of the bridge for failure at the truss/lower chord connection. To make a more conclusive judgment of the ties' value, a more thorough analysis of loading conditions and failure locations would be necessary.

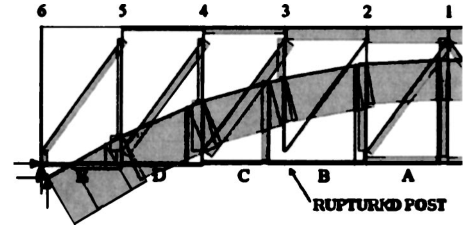

Fig. 10. Axial forces of damaged arch-truss with steel ties due to dead+midspan live load

Fig. 9. Photograph of steel ties in the Pine Grove Bridge

Analysis of the Burr-arch truss concludes that while stresses and deflections for the arch and truss separately exceed allowable limits, in the combined (as-built) system of the Pine Grove Bridge, both stresses and deflections are safely within allowable limits. Defining the relative structural contributions of the arch and truss is an ambiguous endeavor to be sure. Historically, many sagging trusses, or those covering longer spans, had an arch added to them in order to reduce deflections. In a previous study, Kemp and Hall (1975) state that problems with long-term deflections "are largely eliminated with the addition of the arch which transmits its loads by compression in a very direct manner to the abutments." This seems to support the idea of the arch as a stiffening element. However, this study has shown that for realistic loading conditions, the arch carries over *three times* the load of the truss, attesting to its structural dominance. The arch does, however, depend on the truss to counteract the large bending moments and shear forces due to concentrated live loads. Given that dead loads dominate for these structures, we conclude that the arch dominates structurally, while the truss reinforces the arch against concentrated loads. This conclusion depends on a sound arch, springing into solid abutments with good bearing, and positive connection of the arch to the truss.

Town-Lattice Truss

The Town-lattice truss is shown in Fig. 11. The truss supports are modeled as pinned at the left end and roller supported at the right end. The truss is modeled with fixed joints, since all connections contain at least two wooden dowels, or trunnels. Although minute rotations are inevitable, modeling this behavior proved beyond the scope of the study. Dead load of the Town-lattice truss is based on the measured lumber dimensions and a unit weight of 561 kg/m^3 (35 pcf). These loads are placed at joints in a manner that approximates the actual loading distribution. Live loading

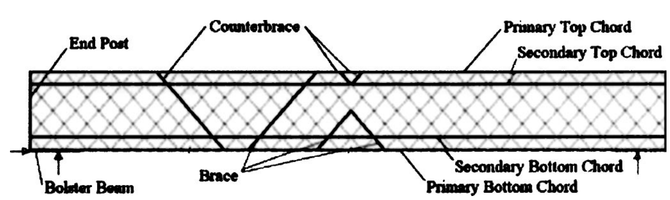

Fig. 11. Centerline model and labeling system of the Town-lattice truss

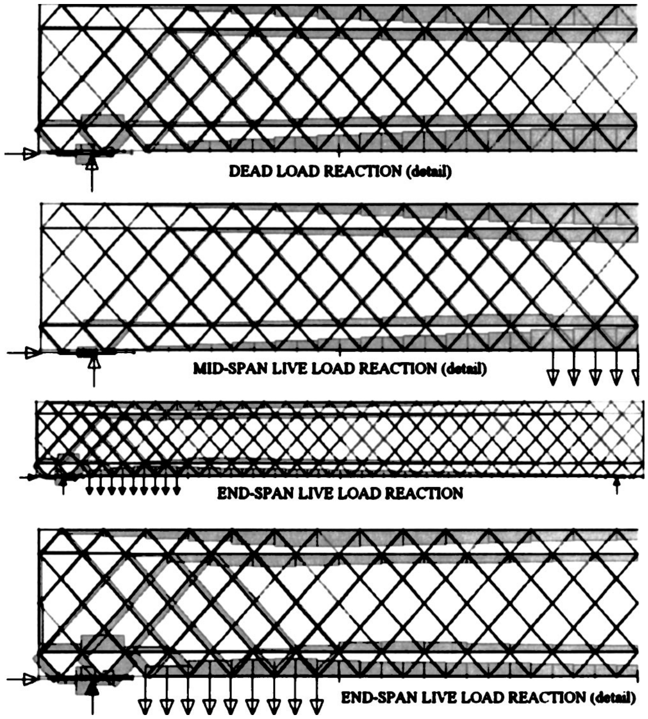

Fig. 12. Axial forces of the lattice truss due to three loading conditions

was modeled to resemble the truck used in field tests. The truck, which the Vermont Agency of Transportation provided, was found to weigh 88.70 kN (19.94 k). This weight was used for the live load in our model, and spread over a distance which approximates the length of the truck. Unfortunately, deflection data gathered in the field was unusable; see HAER documents for further details (2003a).

In typical Town-lattice trusses, the chords are made up of at least two symmetric members, and where one timber ends and another begins there is no connection; rather, the ends are simply butted against one another. Since this cannot transmit tension, these butt joints are staggered so that the remaining continuous timbers, as well as the sister chord, may carry the load. This interesting detail is beyond the scope of our work, and the stresses reported in the chords assume the full area may be considered. However, it should be understood that these stresses might be increased if they occur near the termination points of chord timbers (Pierce 2001).

Shown in Fig. 12 are the axial force diagrams of the Town-lattice truss under dead load, midspan live load, and end-span live load (note that where symmetry allows, only half-spans are shown). To aid in interpreting these diagrams, refer to the shear

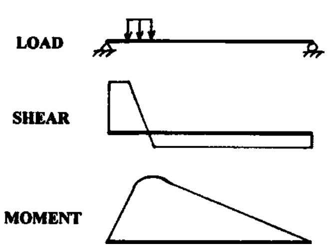

Fig. 13. Shear and moment diagrams of a simple beam under end-span load

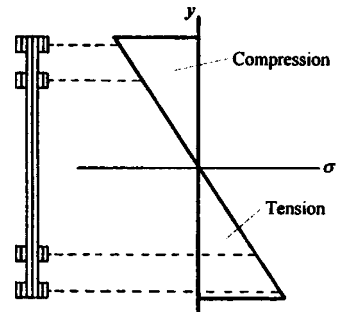

Fig. 14. Stress distribution due to moment in truss cross section assuming uniform beam behavior

and moment diagrams of a simple beam previously provided for uniform and midspan loadings in Fig. 5. Such diagrams for an end-span loaded simple beam are provided in Fig. 13.

The simple beam analogy accounts for the general behavior of the truss, as the global shear and moment demands are manifest in the lattice and chord member forces, respectively (note that the braces and counterbraces are referred to collectively as the lattice or lattice members). The structural behavior of the truss can easily be conceptualized: The chords bear a force couple resisting the global bending moment, and the lattice transmits forces between the chords in order to keep them from shearing or sliding past one another. Likewise, we see in Fig. 12 that under uniform loading, such as dead load, the top chords act in compression and the bottom chords act in tension, the braces are in compression, the counterbraces are in tension, and the end posts see only small compressive forces.

Our Town-lattice truss has both primary and secondary top and bottom chords (Fig. 11). The primary chords carry approximately equivalent forces, as they form a force couple. The secondary top chord carries, at midspan, a force equal to 67% of the primary top chord. Since the secondary chords are exactly two-thirds (0.67) of the distance from midheight to the primary chords, this supports the beam analogy, which defines the stress at any point along the cross section of a beam to be directly proportional to its distance from the neutral axis (center) of the beam, as seen in Fig. 14. For equally sized chords, the neutral axis is in the center, but for chords of differing size the neutral axis location must be calculated. Due to the large weight of the decking, the beam analogy is distorted in the secondary bottom chord, which carries 57% of the force of the primary lower chord as opposed to the expected 67%.

Of course, not all actions are congruent with the beam analogy. There are stress concentrations near the supports as seen most significantly in the lattice members and secondary bottom chord (Fig. 12). The stress of the lattice, here is generally about twice as large at this point than at any other lattice section. The largest stress in the secondary bottom chord also occurs here at the ends

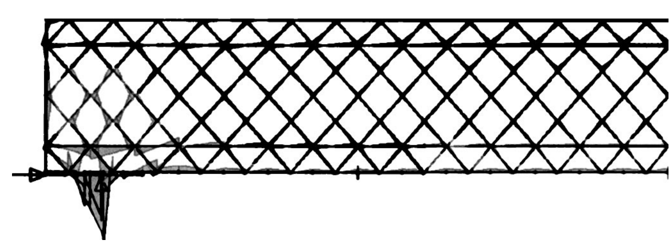

Fig. 15. Moment of the lattice truss due to dead load

Table 6. Maximum Stresses and Deflections of Town-Lattice Truss

Parameter	Dead load	Midspan live load	End-span live load	Dead + midspan live load	Dead + end-span live load
Maximum compressive stress [kPa (psi)]	−9,163 (−1,329)	−1,340 (−195)	−2,340 (−339)	−10,510 (−1,524)	−12,600 (−1,827)
Location	Counterbrace end	Counterbrace end	Counterbrace end	Counterbrace end	Counterbrace, end
Maximum tensile stress [kPa (psi)]	4,380 (635)	979 (142)	1,500 (217)	5,080 (737)	6,430 (933)
Location	Counterbrace, end	Bottom chord, middle	Counterbrace, end	Counterbrace, end	Counterbrace, end
Maximum deflection [cm (in.)]	1.52 (0.60)	0.41 (0.16)	0.15 (0.06)	1.93 (0.76)	1.83 (0.72)
Location	Middle	Middle	Middle	Middle	Middle

Table 7. Maximum Strengths of Eastern Spruce

NDS maximum allowable stress[a]			FPL average strength[b]	
Comp, ∥ [kPa (psi)]	Shear, ∥ [kPa (psi)]	Tension, ∥ [kPa (psi)]	Comp, ∥ [kPa (psi)]	Shear, ∥ [kPa (psi)]
5,340 (775)	450 (65)	5,000 (725)	38,300 (5560)	8,019 (1163)

Note: NDS=National Design Specification; FPL=Forest OProducts Laboratory; and∥=property strength parallel to the wood grain (shear strength parallel to grain is the limiting strength even when loaded transversely).

[a]Values shown do not contain adjustment factors for safety or resistance. Values are an average of those for black, red, and white spruce.

[b]Values for tension parallel to grain are not available for large timbers.

rather than at midspan. Another point of variation from the beam analogy occurs in the lattice and the secondary bottom chord immediately around points of live load application. Tension is induced in the lattice, and compression is induced in the secondary bottom chord. This induced compression in the secondary bottom chord does not generally overcome the global beam behavior of the truss which puts both bottom chords in tension.

Fig. 15 displays the local bending moments of the truss elements under dead load. The maximum moment occurs near the point of support in the primary bottom chord at a magnitude of 19,000 N m (14,000 lb ft). Dead load arising from the floor beams exacerbates the bending in the primary bottom chord. The same moment causes a significant contribution to the stress in the bolster beam at the support.

Table 6 contains the maximum stress values and maximum deflections calculated under the three loading cases. The strength properties of the suspected wood species of the bridge are provided in Table 7 for comparison. Dead load behavior dominates the combination of dead and live load, since the total dead load is about seven times greater than the live load modeled. For all typical loading cases, the largest stresses occur in a counterbrace at the end of the truss. The critical case is the dead plus end-span live loading, with maximums occurring in the lattice rather than in the chords. For the selected member sizes, the critical members in the Town-lattice truss are the lattice members immediately above the first support. Maximum stresses in compression and tension are 136 and 29% greater, respectively, than allowable NDS stresses, but well below the FPL values listed. The largest deflection of the analyzed load cases was 1.93 cm (0.76 in.), or $L/1{,}600$.

The only members under significant shear force are those near the support. The greatest shear stress for the given loadings occurs in the bolster beam at 2,330 kPa (338 psi). This is in significant excess of the allowable NDS limit of 450 kPa (65 psi). However, the NDS makes a specific exception in this case: "Shear design at supports for built-up components… such as between web and chord of a truss, shall be based on test or other techniques," where compressive stress concentrations alter the timber's shear strength (AFPA 1997). Additional work would be required to prove that these high stresses may not be of significant concern.

Structural Efficiency of the Town-Lattice Truss

To further explore ideas of structural efficiency, we will consider the chord sizing, the effects of the bolster beams, the value of the

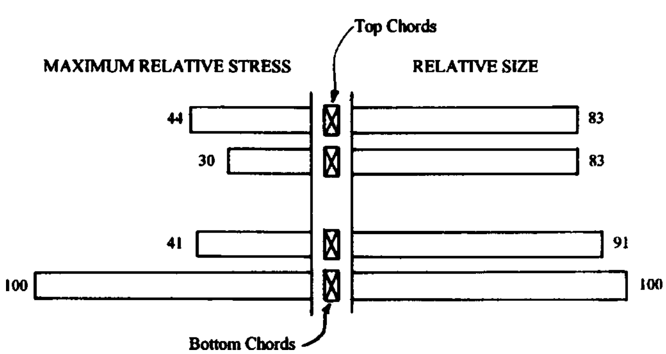

Fig. 16. Efficiency of chord-member sizing

Fig. 17. Photograph of typical bolster beam at Brown Bridge

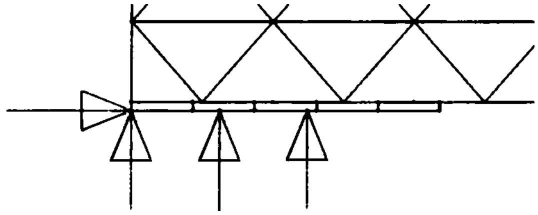

Fig. 18. Centerline model of bolter beam

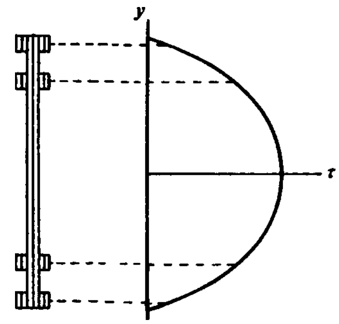

Fig. 19. Shear stress distribution in truss cross section assuming uniform ideal beam behavior

secondary chords, and the uniform placement of the lattice members. Details of the connections are in general not considered here.

Chord Sizing

Consideration of the relative stress versus the relative sizing of the chord members can be instructional in understanding structural efficiency. The maximum relative stress has been found by assuming all chord members to have the same cross-sectional area. Thus, effects of moment are included, and the greatest stress each type of chord received out of all of the loading conditions was found. By plotting these values against the equivalently proportioned actual member sizes, we arrive at Fig. 16. The much larger magnitude of loading in the primary bottom chord compared to the other chords is readily apparent. To have a more efficient size distribution, one would either increase the cross-sectional area of the primary bottom chord, or decrease the area of the other three chords accordingly.

Typically, chord forces follow the beam analogy, thus top and bottom chords should be of the same cross-sectional size. However, our more detailed analysis shows that the primary bottom chord sees significantly greater forces than the other chords, due to stress concentrations at the supports. Interestingly, the designer—Nichols Powers—chose chord member sizes which partially reflect this fact, as the cross-sectional area of the primary bottom chord is larger than the other chords. This suggests that Powers had a deeper understanding of the behavior of the lattice truss than available from the simple beam analogy.

Bolster Beams

A notable feature of this and many other such wooden bridges is the bolster beams at the points of support, as shown in Fig. 17. Although in the current state, the length of the bolster beam cantilever is quite different at each support, for our model a conservative average was used at each support. The bolster beam is modeled with a cantilever length of 114 cm (45 in.) and length of support of 152 cm (60 in.). A detail of the centerline model of the left support is shown in Fig. 18. It was found that only the innermost vertical support acts in compression, the outer two acting in tension. Since there is no tensile connection here, these supports were removed and this analysis has placed only a single support at the innermost position.

By cantilevering from the abutment, a bolster beam helps to diffuse the concentrated forces occurring near the support, namely the large shear forces. To understand their effect, the Town-lattice truss was analyzed without the bolster beams, and key stress values are shown in Table 8. As expected, near the end, large increases in the demands occur. Without the bolster beams in place stresses increase markedly; in the worst individual case (the primary bottom chord), the increase is 53%. Further, the maximum stress in the entire model (a brace near the support) sees an increase of 22% when the bolster is removed. The bolster beams, then, play an important role in reducing the maximum stresses of the structure, which occur near the support.

Secondary Chords

The secondary row of chords (Fig. 11) were added by Ithiel Town to his first patent after many of the originals, containing only primary chords, were "prone to warp" (James 1982). Whether this

Table 8. Maximum Axial Stresses due to Dead + Midspan Live Load With and Without Bolster Beams

	With bolster		Without bolster		
Element	Location	Axial stress [kPa (psi)]	Location	Axial stress [kPa (psi)]	Change (%)
Primary top chord	M	−3,590 (−521)	M	−3,590 (−521)	0
Secondary top chord	M	−2,520 (−365)	M	−2,520 (−365)	0
Secondary bottom chord	M	2,010 (291)	M	2,010 (291)	0
*	E	2,680 (388)	E	2940 (427)	10
Primary bottom chord	M	3,530 (512)	M	3,530 (512)	0
*	E	−5,850 (−849)	E	−8,929 (−1,295)	53
Brace	E	−7,012 (−1,017)	E	−8,936 (−1,296)	27
*	E	−5,920 (−859)	E	−7,212 (−1,046)	22
Counterbrace	E	−10,510 (−1,524)	E	−12,800 (−1,857)	22
*	E	5,080 (737)	E	4,780 (694)	−6
Bolster beam		−6250 (−906)			

Note: Stresses due to the largest axial force are listed initially. Other significant stresses are denoted by an asterisk. The notations "M" and "E" refer to the middle region and end region of the truss, respectively.

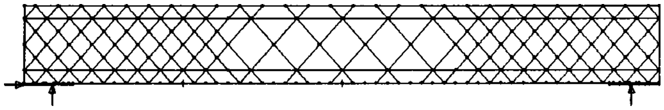

Fig. 20. Model of modified lattice truss

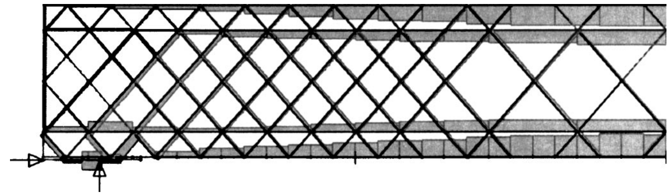

Fig. 21. Axial force diagram of modified lattice due to dead +midspan live load

refers to significant in-plane deflections, out-of-plane bowing, or both, is not known. Interestingly, Town introduced the additional chord members in a secondary row, rather than simply adding more material alongside the original primary chords. Bending strength and rigidity would favor the addition of the secondary chord at a maximum distance from the neutral axis, concurrent with the primary chords. However, a location closer to the neutral axis, as selected by Town, favors increasing shear strength and rigidity (Fig. 19) more directly than bending.

Consider a model of the truss where the secondary chords have been moved to the outside, i.e., added to the primary chords. Loading this single-chord model with the same dead and midspan live load as the original truss configuration, the maximum deflection is 1.63 cm (0.64 in.)—less than the original truss deflection of 1.93 cm (0.76 in.). We also find that the maximum stresses are greater than those of the original truss by about 20%. The locations of greatest stress are in the lattice members, adjacent to the support. These members receive significant axial loads and bending moments, as a result of the large global shear in this area. Therefore, the as-built location of the secondary chord has a positive effect in resisting the global shear of the truss, thus decreasing maximum stresses, although some sacrifice in bending stiffness is made.

There is another more practical side to this issue. If one were to double the size of the primary chords, this would seem to double the global moment capacity of the bridge. However, this is only true if the strength of the trunnels were adequate. To be sure, more trunnels would be needed. However, this presents a problem, since the lattice members already have four holes at their lowest intersection. Moving the additional chord material up to the next lattice intersection seems the easiest solution. In this way, the addition could be accomplished in the same manner as the primary bottom chord, using the same number of trunnels.

Lattice Members

Another question of structural efficiency arises in reference to the lattice members. In all loading cases which included dead load, the largest stresses in the lattice members occurred at the ends. While constructional simplicity favors uniform spacing of these members, it does not seem structurally efficient. To explore this point further, consider a hypothetical alternative design where the generally lower global shear demand at midspan is reflected in the structure by omitting every other lattice member near the middle region of the bridge (Fig. 20). The idea of the alternate design would be to achieve a decrease in dead load, presumably without a significant reduction in strength. An axial force diagram of the system under its approximated dead load and an identical midspan live load as previous analyses are shown in Fig. 21. Larger forces begin to appear in the middle lattice members, but they are still no greater than those at the ends.

Table 9 provides a comparison of the stress demands of the modified and original trusses. In the top chords, the modified system has larger stresses, and in others, most importantly the maximum stresses in the lattice, the modified system has slightly smaller stresses. The deflection is 3% greater in the modified system. This modified lattice has favorable results: The maximum stresses of the system are decreased while the deflection is only slightly increased. Further, less timber is used, and fewer time-consuming trunnel connections are required. Of course, one would have to further examine this modified system with various load cases and removal of different lattice members to gain a better understanding of its value. However, the system is an in-

Table 9. Comparison of Maximum Stresses and Deflections due to Dead + Midspan Live Load for Original and Modified Lattice-Trusses

	Regular lattice		Modified lattice		
Element	Location	Axial stress [kPa(psi)]	Location	Axial stress [kPa (psi)]	Change (%)
Primary top chord	M	−3,590 (−521)	M	−3,770 (−547)	5
*			M	−4,110 (−596)	14
Secondary top chord	M	−2,520 (−365)	M	−2,600 (−377)	3
Secondary bottom chord	M	2,010 (291)	M	2,120 (307)	5
*	E	2,680 (388)	E	2,630 (382)	−2
Primary bottom chord	M	3,530 (512)	M	3,490 (506)	−1
*	E	−5,850 (−849)	E	−5,770 (−837)	−1
Brace	E	−7,012 (−1,017)	E	−6,902 (−1,001)	−2
Counterbrace	E	−10,510 (−1,542)	E	−10,360 (−1,502)	−1
*	E	5,080 (737)	E	5,000 (725)	−2
Deflection [cm (in)].	M	1.93 (0.76)	M	1.98 (0.78)	3

Note: Stresses due to the largest axial force are listed initially. Other significant stresses are denoted by an asterisk. The notations “M” and “E” refer to the middle region and end region of the truss, respectively.

teresting hypothetical modification reflecting ideas of structural efficiency and highlighting some of the issues brought out through our analysis.

Conclusion

As part of a National Park Service program through the HAER, two 19th century covered wooden bridges, a Burr-arch truss and Town-lattice truss, were documented and recorded in the Summer of 2002. Structural analyses of these two bridges were performed to provide insight into the specific bridges studied and the overall structural significance and efficiency of these two bridge forms. The Burr-arch truss of the Pine Grove Bridge is well designed, even by today's standards. Predicted maximum stresses are well below current design standards, and predicted deflections $(L/3{,}300)$ are quite low. The structural behavior of the Burr-arch–truss system is rich and complex. The stiffness of the system is significantly greater than the sum of its arch and truss components. Although it is traditionally thought that the arch was added to the truss as reinforcement, the arch actually carries maximum forces over 350% greater than the maximum forces of the truss, making its structural dominance clear. However, the arch's advantages are only made possible by the truss, for, without the truss, the arch would undergo such large deformations under live loading as to render it useless. The arch provides a direct route to carry loads to the abutments, and the truss provides the moment capacity of its chords for withstanding concentrated forces. Investigations of camber and the addition of steel ties indicate that while these are generally considered to be beneficial elements, analyses of the systems indicate varied effects, which require further study.

In the Town-lattice truss of the Brown Bridge, we find a structural system that acts much like a simple beam. However, significant stress concentrations occur, especially near the supports. Calculated maximum deflections of the bridge are a mere $L/1{,}600$ for the loading cases considered. Considering the chord sizes indicates that the designer understood the stress concentrations that lead to greater demands in the primary bottom chord than the primary top chord. It was also found that the addition of the secondary chords is more favorable than adding the same amount of additional material to the primary chords, from the standpoint of stress demands and ease of construction. An investigation of the omission of selected lattice members from the middle region of the span was used to demonstrate further aspects of structural efficiency relevant to the Town-lattice truss.

The Burr-arch truss and Town-lattice truss are two examples of efficient 19th century covered wooden bridge forms. Still in use today, the Pine Grove Bridge and Brown Bridge, respectively, demonstrate their successful application. Through study of these bridge forms, even simple studies such as those presented herein, we may better understand the behavior of early American wooden bridges and how they were designed. It is hoped that such insight may aid those wishing to better understand these bridges' historic significance, and those who provide historic preservation to these national legacies.

Acknowledgments

The writers would like to thank the HAER and the Federal Highway Administration for the opportunity to research these covered wooden bridges. Without such support, in-depth documentation of these bridges would not have been possible.

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