The Restoration of Covered Bridges

While engineering techniques have gotten more sophisticated and complicated over time, it is beneficial to revisit tried and true techniques in order to preserve valuable artifacts of our engineering heritage.

PHILLIP C. PIERCE

Andrea Palladio, a Venetian architect (1518–1580), is usually credited as the first to describe the form of structure recognized as a truss, as presented in his *Four Books of Architecture*, more commonly referred to as his *Treatise on Architecture*, or simply *Treatise*, circa 1570. Yet some posit that he was really only the first to publish information known to many at that time, including examples constructed (and possibly still extant) in Switzerland. In either event, little attention was paid to his writings until the middle of the eighteenth century when European nations began building the bridges required for significant transportation systems. While France had been the leader in early engineering, based primarily on advances in stone and arch theory and construction, the Swiss and Germans were devoting more attention to using timber trusses in their bridges. Most timber bridges in Europe were not "covered," although the oft-cited Schaffhausen Bridge over the Rhine River, constructed by the Grubenmann brothers in 1758, which included an awkward and inefficient timber roof, was an impressive two-span (171 feet [52.1 m] and 193 feet [58.8 m]) bridge. Many of the other early examples of so-called "covered bridges" stemmed from efforts to provide roofed galleries, usually over simple pile and beam bridges, dating back many centuries.

These early timber covered bridges were somewhat primitive. They consisted of piles driven into the riverbed, with timber beams spanning longitudinally between pile caps. The covers were more for the convenience of users who wanted to linger on a pleasant bridge setting. To span deeper rivers or gorges, eighteenth-century builders found piers to be costly, if not impractical, and they started looking for ways to span greater distances. They did not leap directly to pure truss forms, but passed through some versions of braced beams first. Early German and Swiss truss bridges relied on kingpost and queenpost configurations (with modifications) to add arch action via a strutted beam. Some of the German bridges included diagonal panel bracing in trusses with parallel top and bottom chords. The Swiss often relied on

ever-heavier timber framing, without many diagonal members. They preferred to build up very deep beams, using mechanical connectors between stacked layers — an effort at laminating deep members from smaller members, but without relying on structural adhesives. Other developments in the evolution of timber truss bridges followed in several other European countries, but early bridge building in the United States really led to the most significant advancements in the theory of truss behavior.

Historical Background of Covered Bridges in America

Early Truss Construction in the United States. Americans desiring to travel inland from coastal areas immediately faced the need to span streams of various sizes. Those sites conducive to pile driving were crossed with the classic multiple-span, timber stringer structures. Deeper water demanded longer spans. The gradual developments in Europe provided insufficient guidance to the American pioneers faced with a compelling need to build so many and such demanding structures as fast as they were needed. As might be expected, enterprising and ingenious craftsmen, business people and visionaries forged ahead, willing to test a myriad of structure styles to meet the demand for safe waterway crossings. Some of these structures were obviously modeled after examples in Europe, while others clearly included ideas that were unique to the Americans.

A notable advancement in timber bridge building was the crossing of the Connecticut River at Bellows Falls, Vermont. Colonel Enoch Hale used a two-span structure with a total length of 365 feet (111.3 m). The supporting structure was a strutted beam and took advantage of a natural and striking rock pier in the middle of a natural cascade. The bridge was immediately considered to be a major accomplishment because it was the first to provide spans longer than possible with simple beams.

The First Covered Bridge in the United States. Another American bridge-building pioneer was Timothy Palmer. He was an extraordinarily energetic, talented and prolific bridge builder who experimented with progressively flatter structures that relied less on arch action. The bridges built by Palmer through his career consistently used more panel braced timber frames in configurations that deserve identification as trusses. After constructing several large bridges, Palmer sought and gained approval to span the Schuylkill River at Philadelphia. His resulting structure was substantially different from earlier spans built at the same spot, and included three spans — two of 150 feet (45.8 m) and one of 195 feet (59.4 m) — without struts from below. The trusses were built of heavy timber members with bracing and the bridge was completed in 1805 (or 1806, depending on the source). The bridge cost so much and was so immediately critical to ongoing commerce that it was enclosed with sides and a roof to protect it from weathering — leading to its naming as the "Permanent Bridge." Although there are hints of even earlier "covered" bridges in the United States, this bridge is the one most often so cited.

Patents & Covered Bridges. The United States established its Patent Office in 1790. Tragically, for the purposes of historical research, a fire destroyed the first Patent Office in 1836, with the loss of all patent records to that date. Efforts were made to restore as many of the patents as possible, yet many remain lost forever. Hence, any definitive statements of fact regarding the earliest patents related to the developments of timber trusses and covered bridges are suspect. Not surprisingly, some historians have made heroic efforts to compile as many of the lost pieces as possible. Richard Sanders Allen deserves special recognition for his unpublished compendium of *Thirty-Two Lost Bridge Patents.*

Early American bridge builders actively pursued patents for their designs in an attempt to gain more bridge construction contracts. A few of the very first patents involved general bridge construction, but by 1797 there were several that involved specific schemes for timber arches. Among others, Timothy Palmer received a patent that year (the details of which remain sadly unknown) and he began construction of his Permanent Bridge only a few years after this, his initial patent.

Theodore Burr obtained the first of his eventual multiple patents in 1804 (or 1806, again, the year varies according to the source), which regrettably remains among the unrecovered records. His second patent was issued in 1817. Burr's trademark design dates from this patent. He extended curved lower ribs that had reached only bottom chords, up alongside the trusses, all the way to the top chord. This superposition of arch and truss forms seems to have been influenced by earlier bridges built in Switzerland. The resulting structure has been described as a combination of conventional trusses (parallel chords with compression diagonals) and supplemental arches. One of Burr's early examples of this bridge form, and probably the basis for his 1817 patent, was his Union Bridge crossing of the Hudson River between Lansingburgh and Waterford, New York, circa 1804. This structure was 800 feet (244 m) long, with four spans. It was rebuilt, after being in service for some time, to include a roof and siding. This heavily braced and counterbraced structure exemplified what today is called the "Burr Arch."

Lewis Wernwag was born in Germany in 1769 and obtained a patent (which also remains lost) in 1812. The patent most likely described a structure similar to his crossing of the Schuylkill River at Philadelphia's Upper Ferry. The huge 340-foot (104-m) trussed arch span was quickly termed the "Colossus" and represented a major triumph in bridge construction with its attractive and apparently efficient use of timber, supplemented with iron rod bracing members. Wernwag owned a metal works company and included more reliance on early forms of metal connections and components, rather than on more traditional timber joinery only. Regrettably, the bridge was lost to fire in 1838. He received a second patent in 1829 for improvements to this structure.

Ithiel Town (1784–1844) of New Haven, Connecticut, was a prominent architect known for designing many types of buildings. He also planned many bridges, initially experimenting with various truss arch combinations. However, Town wanted to devise a structure that would require less carpentry skills than required by the intricate joinery details of some of the early bridges. Using only planks joined with round wooden pegs, he began developing a lattice style of truss construction and obtained his first patent in 1820. He was nearly as good a promoter as an inventor and the lattice truss became very popular, although it has been criticized for its apparent waste of material. This truss layout proved to be very adaptable. It could include heavier members for longer spans, and even be "doubled up" to include two layers of web members and three layers of chords for heavy loads, such as those generated by the railroads. A few of his bridges were built with such heavy members that they became identified as a "timber lattice," as compared to the more common "plank lattice." The most famous of the surviving timber lattices is found in the Windsor–Cornish Covered Bridge over the Connecticut River between Vermont and New Hampshire, which remains one of the longest two-span covered bridges in the United States.

Stephen Long (1784–1864) had a varied background and career. He gained his experience as a timber bridge builder while serving in the U.S. Army. Long was commissioned to locate, plan and build the Baltimore & Ohio Railroad. He chose to use a standardized truss for all his spans, with timber counterbraces in all the panels. With the addition of timber wedges at the bearing joints between the posts and diagonals, he found that he had better control over the trusses' as-built geometry. He obtained his first bridge patent in 1830. Subsequent printed materials pronounced that these wedges allowed the truss builders to induce member forces in the trusses that effectively "prestressed" the structure (to employ today's terminology).

Long was prolific in his writings and patents — some charge that he deserves more credit for his recording of bridge history than for his actual technical contributions to the field. The wedge feature is very important, although little, if anything, has been written about its contribution to the strength of the bridge.

William Howe (1803–1852) made a major contribution to the evolution of timber-

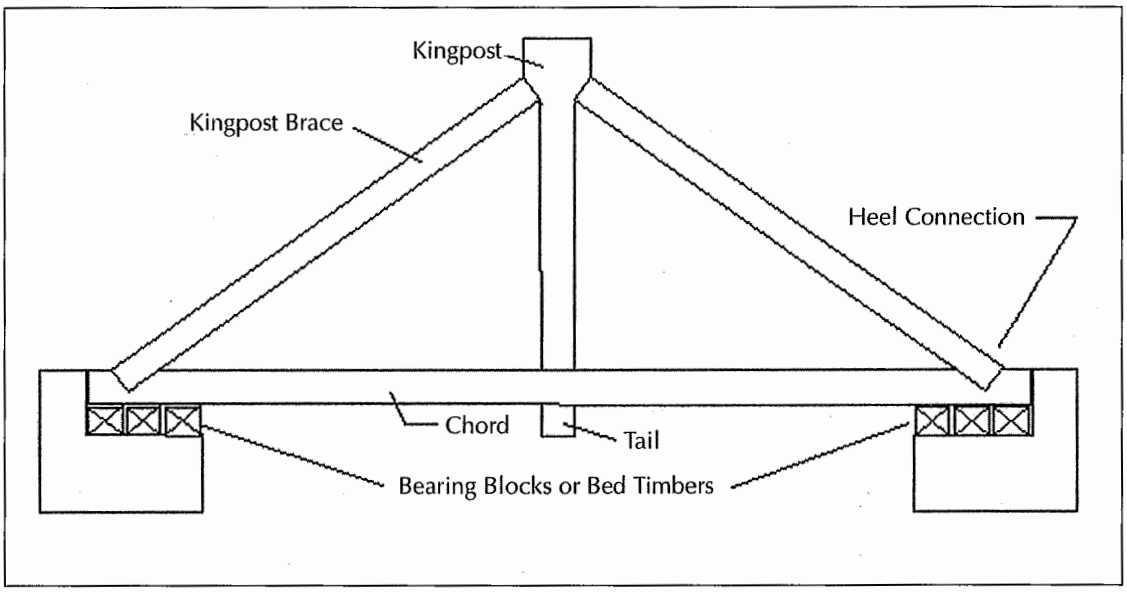

FIGURE 1. Diagram of a kingpost truss.

covered bridges by being the first to use metal components as primary members within an otherwise timber truss. He used timber parallel chords, with timber diagonal braces and counter braces in the panels, but he used round iron rods for the vertical tension members. The threaded rod ends allowed easy adjustment of the structure to keep it tight both during and after erection. Many modifications were made over the years to Howe's original design, in order to address various desired details, but his truss was quickly adopted to withstand the heavy loads on railroads. The popularity of the Howe Truss continues today. It is often selected when constructing new covered bridges. Howe's modification was a major reason for the short life and reduced popularity of Stephen Long's truss — which was essentially the same, but without the iron rod verticals. Howe is also credited with the first published analytical review of his structures, employing the same methods and procedures that Squire Whipple used.

Prevalence, Prominence, Demise & Resurgence. There are many reasonable estimates of the number of covered bridges that have been built in the United States. One very conservative and informed estimate is that at least 10,000 covered timber bridges were built prior to 1900. An examination of the data from the *World Guide* indicates that the largest number of extant covered bridges was built during the 1870s.[1] Subsequent decades saw progressively fewer bridges built. The 1930s seem to end the major construction of covered bridges. Very few were built in the following decades until renewed interest in them developed in the 1960s. This revival of the builders' craft was due, in no small part, to the "last of the covered bridge builders" — Milton Graton. His 1978 book, of the same title, is a fascinating collection of stories and information from his many years spent rehabilitating existing covered bridges and constructing new examples.[2]

Many covered bridges were built in the last few decades of the twentieth century. Interestingly, of the thirty states that currently have covered bridges, over half have built a new covered bridge within the past thirty years; some have built several. And although some owners, engineers, contractors and even bridge users have distinct preferences for truss types, these newer bridges have used nine different truss types.

Summary of Covered Bridge Trusses

The truss types described herein are presented in the order of their span length, starting with the shortest. The first three truss types (king-

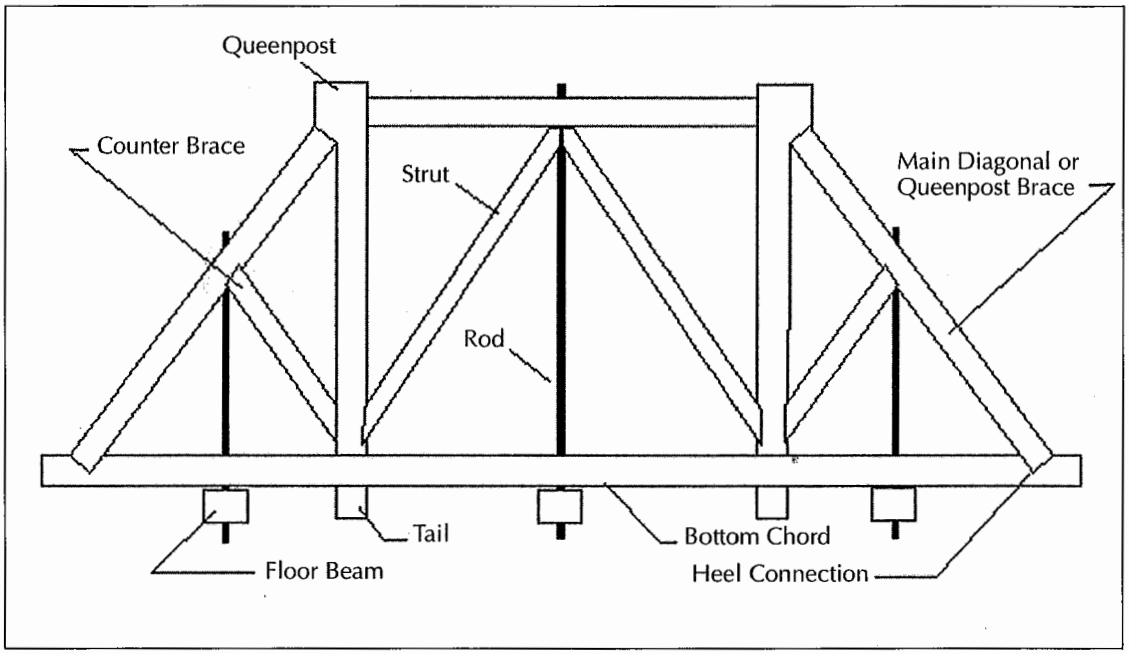

FIGURE 2. Diagram of a queenpost truss.

post, queenpost and multiple kingpost) are ones used in the earliest North American covered bridges. No patents were ever taken on their configurations, and no individual is specifically credited with their development. The other truss types that follow were developed and ultimately named after enterprising early builders/engineers (usually in recognition of a patent obtained for the details of the truss).

Kingpost. The most elementary heavy timber truss configuration is the kingpost (see Figure 1). The inclined members of a kingpost truss serve both as the top chord and as the main diagonals, and resist compression forces. The horizontal member, along the bottom of the truss, is the bottom chord and acts in tension. A central vertical member (the "kingpost") also acts in tension to support the floor loads and serves as the connecting element between the opposing main diagonals. The kingpost truss configuration is said to have two "panels." (A panel is the portion of the truss that lies between any two vertical components.) The span limit for this simplest truss is quite short, typically only about 25 to 30 feet (7.6 to 9.1 m).

The kingpost truss is not very common in the extant United States covered bridge population. There are only about thirty kingpost covered bridges remaining in the United States, with spans ranging from 22 to 70 feet (6.7 to 21.3 m).[1] (It is worth noting that spans of 70 feet [6.7 m] are very unusual for kingpost bridges. Approximately 50 feet [15.2 m] would be the more common upper limit for a kingpost bridge span.) The extant kingpost bridges were built between 1870 and 1976.[1]

Queenpost. The next range in span lengths commonly includes trusses developed from a simple modification of the kingpost. The queenpost truss is, conceptually, simply a stretched out version of the kingpost truss, accomplished by adding a central panel with extra horizontal top and bottom chords (see Figure 2). Classic examples of queenpost trusses do not have any diagonal web members in the central rectangular panel. Therefore, the very simplest queenpost "trusses" are not true trusses at all, but rather frames. The vertical members are termed queenposts. These trusses are considered to have three panels.

There are approximately 101 bridges supported by queenpost trusses, or a little over 10 percent of all the surviving covered bridges in

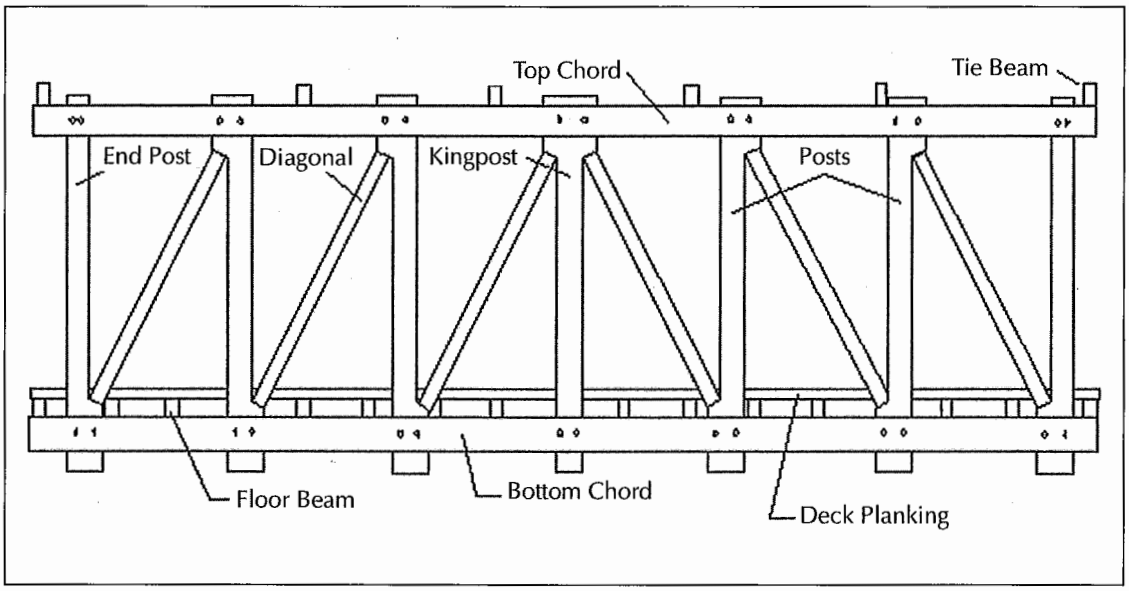

FIGURE 3. Diagram of a multiple kingpost truss.

the United States. Their spans range from 25 to 130 feet (7.6 to 39.6 m) and they were built between 1845 and 1985.[1] The longer span requires that many of their bottom chords have to be spliced longitudinally from separate timbers. This tensile connection is another area of weakness in the truss.

Multiple Kingpost. A straightforward way to stretch the span capability of the queenpost truss is to add panels to the kingpost truss in order to create what is known as a multiple kingpost truss (see Figure 3). Accordingly, the basic kingpost truss is sometimes referred to as a simple kingpost truss. Most of these trusses were built with an even number of panels so that all the diagonals are in compression and all the verticals are in tension under normal loading. A very few multiple kingpost trusses have an odd number of panels, with opposing (or crossing) diagonals in the center panel.

There remain about 95 bridges using multiple kingpost trusses, or a little over 10 percent of all covered bridges in the United States.[1] Multiple kingpost trusses have spans that range from 36 to 124 feet (11.0 to 41.1 m), and they all seem to have been built between 1849 and 1983.[1] Interestingly, in comparing the span ranges and the construction dates between queenpost and multiple kingpost trusses a certain similarity in these two features may be observed.

Burr Arch. Theodore Burr obtained the first U.S. patent issued for a specific timber truss configuration in 1806. The Burr Arch is basically a combination of a typical multiple kingpost truss with a superimposed arch (see Figure 4). The addition of the arch was intended to allow heavier loads on the bridges and to stretch their span capabilities to greater lengths. Surviving examples of Burr Arch bridges have spans of up to 222 feet (67.7 m).[1]

Burr's development was immediately popular with bridge builders and has proven durable. More existing North American covered bridges use the Burr truss than any other type. The classic, or conventional, Burr Arch supports the ends of the arch components at the abutment, with no connection between the bottom chord and arch as they pass by one another (the chord is supported by the abutment separated directly from the arch end). A variation of the Burr Arch (sometimes referred to as a "modified Burr Arch") terminates (and ties) the arch with a connection directly to the bottom chord, which is supported on the abutments.

The arch part of most Burr arches is in pairs, which sandwich a single multiple kingpost truss between them. The most common

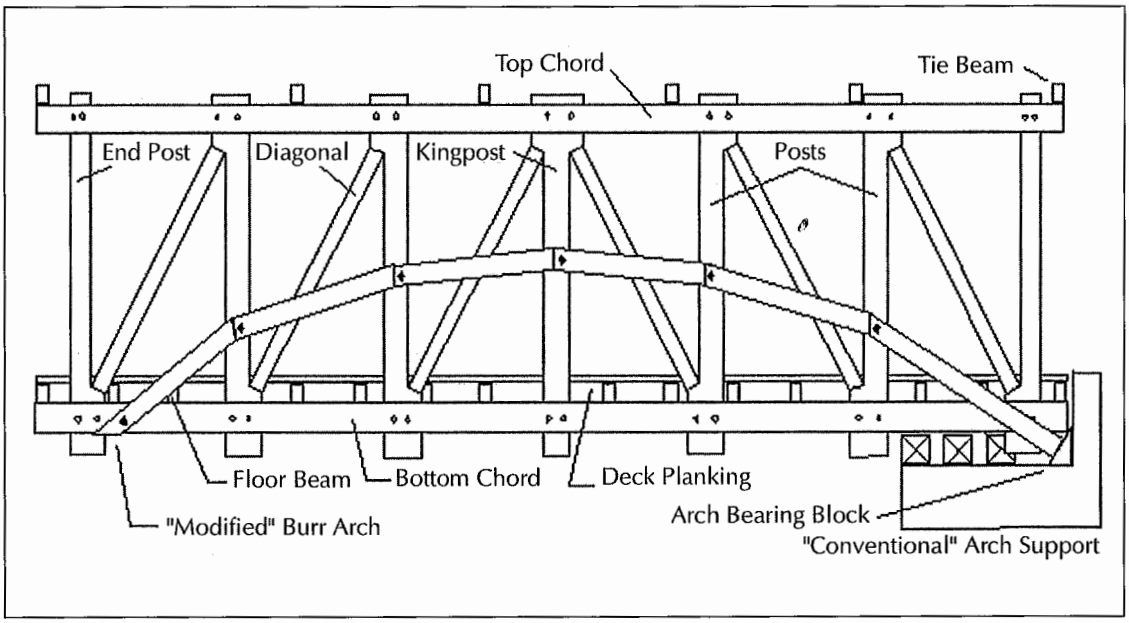

FIGURE 4. Diagram of "conventional" and "modified" Burr arches.

connection uses a single bolt to join the arches to one another through each of the vertical members of the truss. Load sharing between the truss and the arch components is largely dependent on the relative stiffnesses of those bolts. The floor beams carry the live loads to the truss bottom chords and the roof loads bear on their top chords. In order for these vertical loads to be distributed into the arch, the bolts must resist significant vertical shear forces. The initial, traditional Burr arches used arch components sawn from large, single timbers that were lap-spliced to one another at the verticals. Later, the use of continuous, but laminated (multiple-layer) timber arches became popular with some builders.

There are about 224 remaining bridges supported by Burr arches and its multiple variations, or about 25 percent of all covered bridges.[1] The Burr Arch has individual spans that range from 33 up to an impressive 222 feet (10.0 to 67.7 m), the longest being 10 percent longer than the next rival configuration of truss (the Howe). The extant Burr arches were built between the early 1800s and 1988.[1]

Town Lattice. Ithiel Town, an architect by education, obtained his first patent for a unique type of timber truss in 1820 (see Figure 5). All the other trusses discussed above principally rely on large and heavy timbers that require skilled artisans to properly craft the rather elaborate joinery between the various components. Town sought a means of constructing bridges that would rely far less on skilled labor. He also wanted a bridge that could be built from an easily adapted design. His patented truss developed a configuration that could be extended to a wide range of span lengths with relatively little modification of the configuration. Town is credited with a popular maxim that his truss could be "built by the mile and cut off by the yard." In the opinion of many informed bridge aficionados, his patented truss represents arguably the most important development in the history of covered bridges and one that remains a popular and enduring style.

Town's lattice configuration relies on an assemblage of relatively short and light planks that were available and easy to handle. He connected the overlapping intersection of members with round timber dowels or pegs, termed "treenails" — pronounced "trunnels" (and so spelled hereafter). The plank intersections in the web may have from one to three trunnels. Where chord members intersect with web or lattice members the overlapping zone may contain as many as four trunnels. The

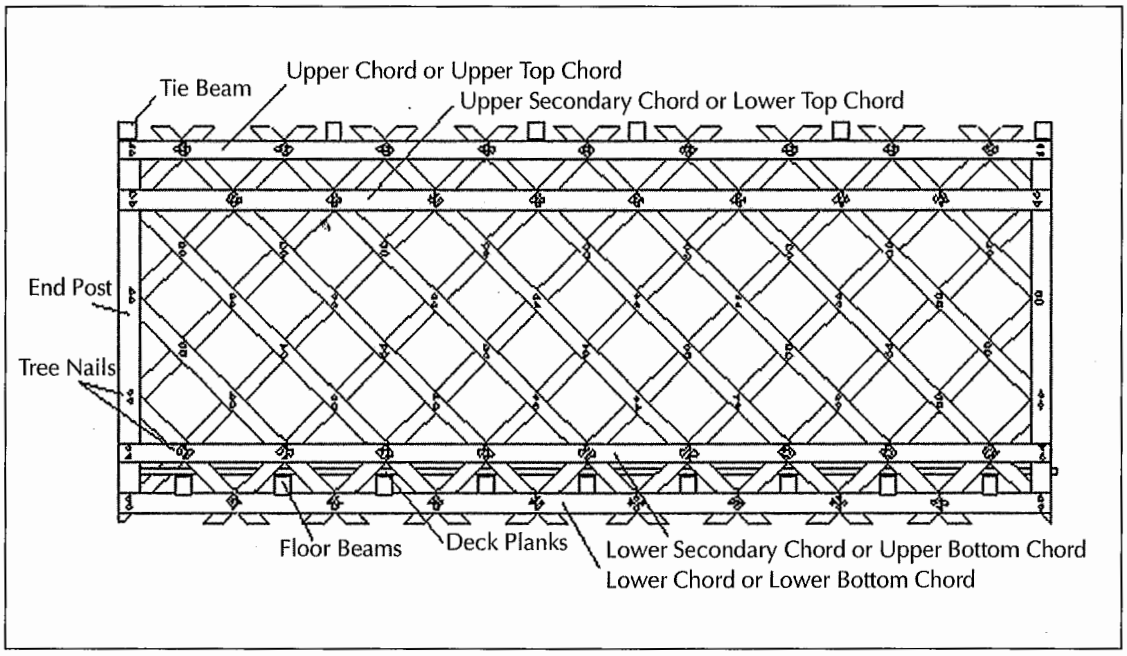

FIGURE 5. Diagram of a Town lattice truss.

dowels are often 1.5 to 2 inches (38 to 51 mm) in diameter. The parallel and closely spaced web members are joined to chords along both the top and bottom of the trusses. Two levels of chords are commonly used as the bottom chords. The top chords may have one or two levels of members. The lower-most bottom chord provides the seat for the transverse floor beams.

Town, or "lattice," trusses are most commonly comprised of thin members with pairs of chords on each side of the lattice webs. In this case, the truss is sometimes termed a "plank" lattice. The chord members are not generally spliced to abutting pieces at their ends, but the terminations are staggered so that any panel of chord has at least one unspliced member. A few Town lattice trusses were fabricated of heavier components using single chord members on each side of the lattice. In this case, the truss is termed a "timber" lattice. The chord members require splices at their ends.

There remain about 135 bridges supported by Town lattice trusses.[1] Town lattice trusses support varying span lengths, from relatively short (only 25 feet [7.6 m]), up to some of the very longest covered bridge spans in the world. Individual Town lattice trusses range in span up to 162 feet (49.4 m).[1] The oldest surviving Town lattice bridge (the Halpin Bridge in Addison County, Vermont) was built about 1824.[1] New examples of Town lattice covered bridges are still being built.

Long Truss. Colonel Stephen H. Long first patented a truss configuration in 1830. His focus was on a parallel chord truss, made with heavy timbers and with crossed diagonals in each panel (see Figure 6). A special feature of his bridge included the use of timber wedges at the intersections of the chords, posts and diagonals. The wedges were purported to allow builders and maintainers to adjust the shape of the panels, and provided the opportunity to adjust the initial camber.

Today, some have further proposed that the wedges allowed builders to induce forced loads in the diagonals in a way that would be described as "post-tensioning." While the concept seems logical when viewing a simple two-panel truss or kingpost configuration, the logic breaks down when considering longer structures with multiple panels. There is no universally accepted evidence that these prestressing forces can be predictably induced.

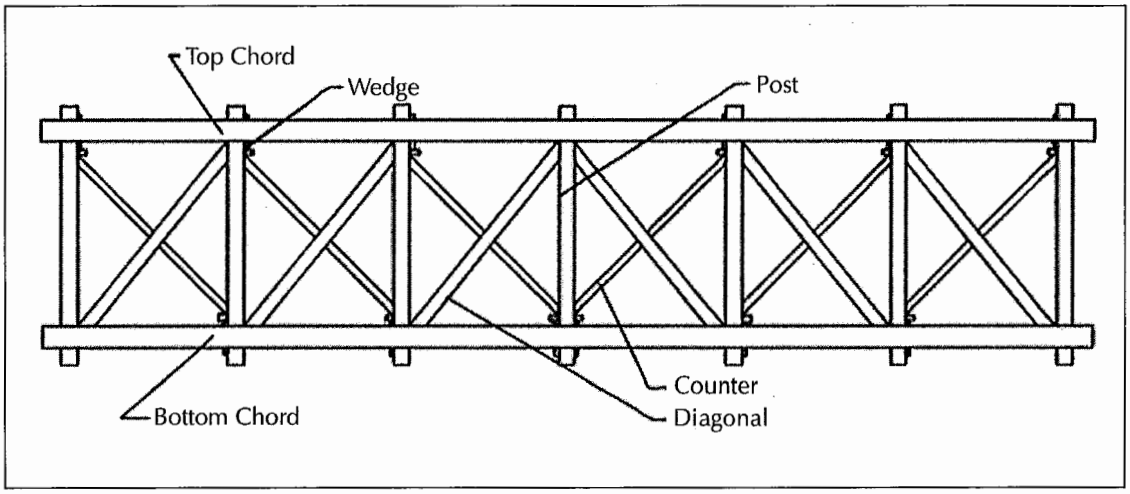

FIGURE 6. Diagram of a Long truss.

However, the wedges do offer an important increase in the strength of the connection between the horizontal component of the load in the diagonal and the chord. The transfer of load without wedges flows from end bearing on the diagonal to cross grain bearing in the post and then from cross grain bearing at the shoulder of the post back to end grain bearing at the shoulder of the chord. The introduction of the wedge distributes the bearing load from the chord over a much larger area of the post via the wedge in direct cross grain bearing.

The Long truss was adopted by many builders for use in highway and railway bridges, but the timing of its introduction meant that it was destined to be quickly overtaken in popularity by the Howe truss. There are about 40 surviving bridges supported by the Long truss, with individual spans that range from 51 to 170 feet (15.5 to 51.8 m).[1] The oldest extant Long truss was built in 1840 and the newest was built in 1987.[1]

Howe Truss. William Howe (1803–1852), of Massachusetts, was granted his first truss patent in 1840 and a second one later in the same year. His second patent used metal rods as the vertical members of what was otherwise a simple timber parallel-chord, cross-braced truss. This patent was the first one granted with some major structural components made with metal. The configuration used easy-to-erect and readily prefabricated components that could be assembled on site and adjusted via threaded connections at the rod ends. Little skilled labor was involved in assembling and erecting this truss type and it became an immediate success (see Figure 7).

The Howe truss is second only to the Burr Arch in popularity of the extant covered bridges in the United States. There are about 143 bridges supported by the Howe truss, or about 15 percent of all covered bridges.[1] The Howe truss has individual spans that range from an unusually short 20 feet (6.1 m) up to an impressive 200 feet (61.0 m), which is only 10 percent shorter than the longest Burr Arch.[1] The oldest extant Howe truss was built in 1854 and the configuration remains popular with new authentic examples built today.[1]

Work on Covered Bridges Over the Last Fifty Years

After covered bridges started to lose favor to metal trusses, there was a progressive loss of knowledge of the nuances of timber design and covered bridge detailing, in particular among the engineers and contractors challenged to maintain the bridges. The covered bridges represented an impediment to "progress" in a time of increased attention to the number, size and weight of vehicles. As a consequence, the extant covered bridges were not maintained and usually replaced by more modern bridges.

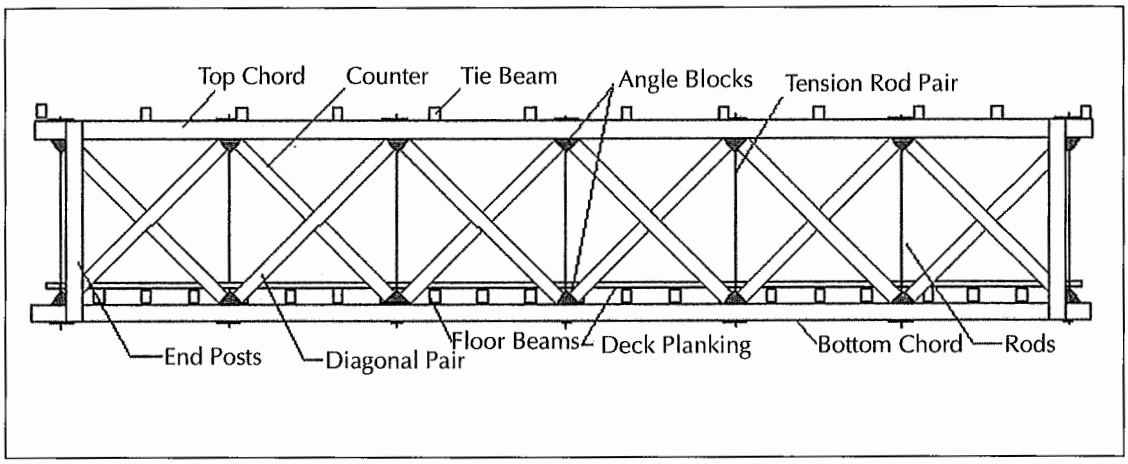

FIGURE 7. Diagram of a Howe Truss.

At one time, there may have been in excess of 10,000 covered bridges in the United States. By the mid-1900s many of those bridges had been destroyed by intentional replacement, arson, flood or neglect.

Unfortunately, when covered bridges were retained and demanded attention, many "repairs" were planned and performed by inexperienced engineers and contractors. Often, steel plates and bolts were installed as an attempt to deal with the deterioration of timber components and joinery. These repair practices mimicked those used on metal trusses. Although well intentioned, the combination of dissimilar materials led to condensation inside the wood, masked by the steel plates, and additional deterioration of the timber was inevitable and hidden from view. In fact, these repairs often made the subsequent condition of the covered bridges even worse for those who followed. As time marched on, the number and condition of covered bridges gradually declined.

The person often cited as a leader in the efforts to salvage remaining covered bridges was Milton Graton — a contractor in New Hampshire. Coming from a long line of carpenters and craftsman, he traveled far and wide to repair bridges and occasionally built new bridges using authentic practices and materials. His work involving covered bridges developed in the 1960s and continued until his death in 1994. His family continues the tradition. Graton's book — *The Last of the Covered Bridge Builders* — offers information about these special structures from his perspective as a contractor.[2]

One of the states that has served as a leader in the resurgence of interest in covered bridge preservation is Vermont. The state is home to about 100 historic bridges. In 1992, the Vermont Agency of Transportation sponsored an in-depth evaluation of 75 bridges owned by individual towns of Vermont. The study resulted in drafting individual plans for each bridge that proposed an identified means of preserving the bridge indefinitely. The plans ranged from rehabilitating the bridge for light or moderate traffic, to bypassing it with a new bridge or moving it to a preservation site.

During this same period of increased attention to the needs of covered bridges, Vermont's U.S. Senator James Jeffords sponsored legislation that was ultimately adopted in the National Historic Covered Bridge Preservation Program as contained in the Transportation Equity Act for the 21st Century, which was passed in 1998. It has provided over $30 million for research on and the rehabilitation of over 100 covered bridges. The challenges of covered bridges continue and additional funding is proposed in the reauthorization of Federal funding for transportation, still waiting for passage by Congress.

Interaction Between Preservationists & Engineers

Historic preservation principles are aimed at maintaining the components of the structure

("its fabric") and the context of the material. Hence, for covered bridges, it is important to retain the form of the supporting trusses and the look of the structure.

Those who take the historic preservation perspective would prefer that repairs be made with as little change from the original materials as possible. If an end of an element is rotted beyond salvage, then its removal and replacement with the same type of material (species and size) would be preferred — limited in nature to only the damaged material and not by replacing the entire element. There is a hierarchy established to deal with such situations, as published and identified as the U.S. Secretary of Interior's Standards for Historic Preservation. In theory, these standards are admirable and represent good practice. In practice, they can at times be applied as intended with success. However, there are other issues that must be addressed as well.

The engineer who is challenged to provide a structure with acceptable reserve capacity for safe passage of live load (be it vehicles or pedestrians) must deal with current knowledge and standards. Most extant historic covered bridges were built with little "engineering" and have withstood the tests of time due to careful construction by skilled craftsman who built according to practice and experience. The vehicles in use at that time little resemble today's traffic.

Wood is widely used in modern construction, yet the use of "heavy" timber in trusses is no longer common and not often included in the educational training of today's engineers. Those few engineers involved with the relatively rare covered bridge (there are less than a thousand covered bridges in the collection of about 600,000 bridges in the United States) must in large part deal with the challenges without much theoretical reference material.

It is common that analytical evaluations of covered bridge trusses indicate substantial theoretical weaknesses that may not be compatible with physical evidence in the field. Many bridges routinely support traffic while calculations indicate that the bridge would collapse. There are several reasons for this odd situation, among them are:

- Current codes were developed for the design of new structures. Intentional or accidental conservatism is not a major problem in that case; it simply requires a slight increase in the sizes of elements. However, those same codes, when used for analytical review of historic covered bridges, may indicate the need for the unnecessary replacement of elements.
- Loading conditions may be unnecessarily conservative or inappropriate.
- Analytical techniques may be inappropriate or inaccurate.
- The assumptions regarding section or material properties of the structure may be conservative.
- Inadequate evaluation of connections and the differentiation between primary element stresses and connection capacities.

Hence, engineers must deal with these sorts of covered bridge conundrums. It is the rare bridge that can be rehabilitated without residual overstresses in at least some components. The experienced covered bridge engineer exercises practiced judgement to decide what overstress is tolerable. Therefore, the decision as to what components will be replaced hinges on the compromises between the demands of safety versus the desires for the retention of a historic fabric.

Related to the debates about what the degree of work that will be done to an historic bridge is the need for clarification about terminology. A good reference for clarification of this terminology is an Internet site hosted by the Advisory Council on Historic Preservation.[3] An important distinction is made among various types of anticipated preservation work — rehabilitation, reconstruction and restoration. According to that site:[3]

> *Rehabilitation (treatment)* — the act or process of returning a property to a state of utility through repair or alteration which makes possible an efficient contemporary use while preserving those portions or features of the property which are significant to its historical, architectural and cultural values.
>
> *Reconstruction (treatment)* — the act or process of reproducing by new construc-

tion the exact form and detail of a vanished building, structure, or object, or any part thereof, as it appeared at a specific period of time.

Restoration — the act or process of accurately recovering the form and details of a property and its setting as it appeared at a particular period of time by means of the removal of later work or by the replacement of missing earlier work.

Restoration and reconstruction are actions that focus on preservation with less consideration of the structural needs of the work. Rehabilitation allows less intensive preservation and more consideration of structural needs.

For covered bridges that are still used by vehicles, the structural demands often require strengthening that alters the original construction. That type of work usually involves rehabilitation rather than restoration. Consequently, rehabilitation may result in the retention of less original fabric. A covered bridge project especially benefits from a close working relationship between the engineer and preservationist to develop better appreciation of both sides of this controversial and contentious challenge.

Critical Engineering Issues Related to Covered Bridges

Self-Weight (Dead Load). In most short- to mid-span concrete or steel bridges, the stresses induced by the weight of the design vehicle represents a large proportion of the total stress in the primary components. Stresses from the self-weight of these bridges, while not small, are usually much smaller than those caused by the design vehicle.

Covered timber bridges, however, are unusual in that the stresses from their own self-weight represent a significant part of the total stresses. It is not uncommon that the chord forces due to dead load are equal to, or even exceed, those caused by vehicular loads. This phenomenon is due, in part, to the weight of the roof and siding that more conventional bridges do not have. The heavy timber trusses also have relatively high weight-to-strength ratios, compared with the efficient beam cross sections used in steel and concrete. Therefore, it is very important that the prediction of a timber-covered bridge's self-weight be as accurate as possible.

The American Association of State Highway and Transportation Officials (AASHTO) Bridge Specifications suggest using 50 pounds per cubic foot (pcf) (800 kilograms per cubic meter [kg/m^3]) for the density of wood, when determining a timber bridge's dead load. The value of 50 pcf is conservatively high and prudent for most common (i.e., "uncovered") bridges and with most (but not all) timber species. (For comparison, many building designs use a density of 35 pcf [560 kg/m^3] as a default density.) The 50 pcf value was established when the specifications were first published in the 1930s. It is strongly influenced by open timber structures that are prone to high moisture contents. Also, creosote wood preservative was commonly used, which could add up to an additional 10 pcf to the weight of the wood.

Hence, a prudent first step in the analysis of a covered bridge is the use of 50 pcf (800 kg/m^3) for the assumed weight of the wood, based on the guidance of AASHTO. If the results of that assumption are acceptable, as evidenced by a calculated satisfactory live load capacity, then no further effort regarding the weight of wood is necessary. However, if the results of the analysis indicate an unacceptably low capacity for live load, then further effort in the determination of the weight of the wood may be warranted.

Wood density is strongly influenced by its moisture content, which can vary widely with environmental conditions. AASHTO's suggested density recognizes the potential for higher moisture contents (and densities) when timber components are exposed to direct wetting from rain. Covered bridge timbers, however, have lower timber moisture contents (the very purpose of the bridges' covers) and the associated reduced timber densities. The bridges inspected in the Vermont study contained timbers with moisture contents much lower than 19 to 20 percent (a commonly cited upper limit of "dry" wood in the timber industry.) The aged and protected air-dried softwood timber found in most covered bridges typically has unit

weights ranging from 26 to 38 pcf (417 to 609 kg/m^3).

This issue is extremely important to the accurate evaluation of the capacities of covered bridges. It is common that analyses prepared using the standard 50 pcf (800 kg/m^3) density would indicate the need to rehabilitate the bridge by replacing existing elements with higher-strength grade timber, unusually large timber components or even non-timber components. In some instances, the bridge would be unable to support any design live load while providing the necessary factor of safety. However, the use of site-specific wood densities usually leads to a substantial reduction in the dead load forces and stresses (up to a 30 to 40 percent reduction is common).

Taking into account design wood density should spur the use of *site-specific* unit weights for the evaluation and rehabilitation of historic covered bridges. The selection of unit weights should be based on standard timber references, such as:

- *Timber Construction Manual,* published in 1994 by the American Institute of Timber Construction;
- *Timber Bridges — Design, Construction, Inspection, and Maintenance,* published in 1990 by the United States Department of Agriculture, Forest Service; or,
- *Wood Handbook,* published in 1999 by the United States Department of Agriculture, Forest Service.

Obviously, defining weights requires determining the species of the various elements, which must be determined for allowable stress purposes in any case. If some components (for example, the floor planking) are made of a more dense species, then unit weights appropriate to that component should be used, resulting in an overall weight of the bridge as a summary of the individual components.

Furthermore, the site-specific unit weight should be based on a reasonable estimate of moisture content. A moisture meter can be used to determine the actual moisture content of representative elements of the bridge. Those elements below the surface of the deck may have higher values, especially if the bridge is relatively close to water. Or, as an alternative (although not nearly as convenient), actual samples of the wood from the bridge can be obtained and tested in a laboratory to determine the actual moisture content. As noted above, it is common to find moisture contents below the 19 to 20 percent threshold that is often cited as the difference between "dry" and "wet" wood. The 50 pcf (800 kg/m^3) density may safely be used in lieu of site-specific densities, should that be desirable.

A related topic involves installing dry, versus green, new timber components during the rehabilitation of historic bridges or for the design of new covered bridges. Conventional practice requires installing only dried primary structural components that would, therefore, have unit weights as discussed above. However, the relatively non-structural components (siding, roof boards and some of the bracing) might reasonably be installed green. While doing so could result in a heavier load than that assumed by the designer, many judge it to be acceptable because those components will dry quite rapidly and soon reach the reduced unit weight, in most cases prior to the bridge being opened to traffic. If one intends to use green timber for bigger, primary components, then the conscientious designer will make appropriate modifications to the unit weight because the larger members will take longer to dry.

Another minor wood density consideration relates to the extra weight associated with most modern pressure-applied preservative treatments. Although usually quite small, common treatments could add another 1 or 2 pcf (16 or 32 kg/m^3), except creosote adds substantially more.

Vehicular Loads (Live Load). A primary design issue, when working with any traffic-carrying bridge, is the selection of the design vehicle. The AASHTO Standard Specifications for Highway Bridges identifies three types of design vehicle load. The first two represent actual individual vehicles and are routinely referred to as the H or HS truck. The H truck configuration includes only two theoretical axles and is intended to represent dump truck type vehicles. The AASHTO specifications

present information related to two sizes of H-type vehicles — the standard 20-ton (i.e., the H20 truck) or a smaller 15-ton vehicle (the H15).

The conventional semi- or tractor-trailer type vehicle is identified by AASHTO as an HS truck configuration. It is identical to the H truck, but with an extra and heavier axle representing the rear axle of the trailer. The standard HS20 vehicle weighs a total of 36 tons, and the smaller HS15 weighs 27 tons.

The third type of design-vehicular load is what AASHTO terms "lane load." This uniform load scheme is meant to represent a string of closely spaced H15 single trucks (with 30 feet between the rear axle of one vehicle and the front axle of the following vehicle), with a heavier H20 truck in the middle of the string. This type of vehicular load is important for longer span structures, where slow traffic can lead to a "bunching effect," with heavier load than that generated by higher speed traffic traveling with more space between vehicles.

These three vehicular load types have evolved from the initial AASHTO specifications published in 1935. While each load type is a much-simplified representation of the diverse vehicle configurations and weights that actually travel the roadways, this trio of AASHTO loads has been retained as acceptably accurate for the purposes of the design of most bridge components. Consistent design loads, unless they are completely unrealistic or non-conservative, serve the structural design profession well and in many ways.

Bridges built in the nineteenth century would not have been designed for these modern vehicular loads. These bridges were crossed by horses and wagons, along with whatever load could be pulled through the openings of the bridges. A load of loose hay was not overly heavy, but skids of logs moved on icy winter roadways could be quite heavy. The commonly anticipated maximum vehicle load, at the time when the original covered bridges were built, would have been significantly less than the single-vehicle H15 load described above. The relatively high proportion of dead to total load in covered bridges has worked in their favor, however, as the live loads have increased over their functional lives.

Most original covered bridges have been upgraded to safely support less than the standard live loads. The consideration of more "community type" vehicles is appropriate in many instances — for example, oil trucks, loaded snowplows, school buses or emergency equipment. A commonly selected design load is H15 to simulate such vehicles; occasionally an H20 vehicle is specified as the minimum desired capacity.

Many covered bridges have been intentionally posted with a lower weight restriction than the bridges could (and often do) safely support in an active attempt to limit the number of heavier vehicles using the bridge. This action tends to reduce the accumulated deterioration of the bridge from overweight vehicles and extends the time between required rehabilitations. In many cases, the more important rationale for the load restriction is for community planning purposes. Regardless of the reason, the design vehicle should be selected to be as small as politically feasible to minimize the potential abuse with heavier vehicles, even if they might safely cross the bridge.

Selecting the design live load vehicle is usually very dependent on the site of the covered bridge. A covered bridge that provides sole access to a dead-end road must be able to support a more diverse and complete set of vehicles than will a covered bridge that can be easily bypassed. Likewise, a covered bridge immediately adjacent to the most traveled roadway network will be used by more vehicles of any weight than would one in a more remote location.

For bridges located in more remote areas, there will be more violations of weight restrictions; hence, bridge designers and owners must carefully weigh whether relying on a load posting is prudent. If regular and excessive load violations are anticipated, then closing or rehabilitating the bridge should be considered.

Snow Load. Most newer, conventional, deck-type bridges are not designed for the weight of snow because standard practice involves removing snow from the bridge with plows.

Typically, it is assumed that the bridge does not have to carry both heavy snow and heavy vehicles at the same time.

On the other hand, covered bridges might have snow on the roof at the same time vehicles pass through it. The bridge would therefore have to support both snow and vehicular loads. In northern states, design snow loads can become quite heavy — 50 psf (800 kg/m^3) or more (sometimes much more). In some of those states, covered bridges are known to carry snow load on the deck level that is deliberately plowed there to allow the passage of the locally prevalent snowmobile. This thin layer of snow represents a light load, which reasonable analysts might neglect. The melting snow, however, may cause some decay in deck level timber components, so those bridge owners might be more particular about regular inspections if they pursue this plowing policy.

However, because the AASHTO bridge specifications do not address this issue, it is up to the covered bridge design engineer to select a prudent snow load and live load combination. Many engineers, experienced with covered bridges, believe that it is too conservative to simply assume that a covered bridge must support both a full-weight design vehicle and full-weight snow load simultaneously. Vehicles passing across a covered bridge vibrate the bridge and cause heavy accumulations of snow to slide off the roof, assuming that the bridge has a common 6:12 pitch or steeper roof. If the bridge has a fairly flat roof, then this snow shedding will not occur and designing for both loads could be prudent. Most building design codes differentiate between a flat roof and a sloped roof (where the latter has a slope of more than 5 to 30 degrees, depending on the specification) when it comes to determining design snow loads.

The bridge's tendency to shed snow load is also a function of the roofing material. A metal roof will shed snow load much more readily and reliably than a roof with wood shingles. Bridges in heavy snow areas would therefore benefit from having a metal roof. Metal roofing systems offer an additional benefit since they are lighter than other types of roofing systems.

If the bridge is closed to vehicular traffic, then full snow load should be anticipated and evaluated. The snow can become quite heavy and can represent a significant load on the structure — in some cases much more than the weight of a design vehicle.

The design snow load magnitude is addressed in many specifications, but one quite commonly cited is the *ANSI/ASCE 7 — Minimum Design Loads for Buildings and Other Structures.* Many state governments have special snow load maps to provide additional guidance on appropriate design snow loads — most of these are targeted toward building design, but are appropriate for covered bridges.

Material Properties/Allowable Stresses. Determining the appropriate material properties and allowable stresses to use in evaluating an extant covered bridge is challenging. The current (2001) American Wood Council National Design Specifications (NDS) contain allowable stresses that are based on extensive information and experience with timber. However, there remain concerns that the NDS values are quite conservative when applied to the timber found in historic covered bridges.

The first step in determining member capacities involves identifying the wood species. The bridge analyst may find several species within an extant covered bridge, so it is important to determine the wood species for the particular structural members under consideration. Unless one is a trained expert in wood identification, it is appropriate and relatively inexpensive to obtain small samples of the members and have them identified by a wood scientist.

The next step in assessing a member's capacity involves determining the "grade" that most accurately describes the members. This identification becomes more problematic than the simple species issue. Again, it is appropriate to seek assistance from an expert. There are several popular timber grading associations, many of which offer services involving on-site and in-situ grading of timbers. As an example, the Northeast Lumber Grader's Association (NeLMA) can provide a grader who will be able to visit a covered bridge and provide a reasonable best profes-

sional opinion about the timber grading characteristics for the critical elements. The cost for this service is relatively nominal compared to the entire analysis cost and should be considered an important aspect of evaluating the bridge. The analyst is cautioned, in advance, that the grader can only provide grading for what material is visible. Also, it must be noted that the grader cannot categorically identify a structural component as being definitely of any certain grade if portions of the member are hidden from view. However, the grader can determine what the member is *not*, based on the member faces that are visible — for example, "the member does not meet the criteria of a Select Structural member" or "it does not meet the requirements for a Number 1 grade."

Therefore, the engineer must still exercise some judgement in this matter. For instance, if a given member does not meet a select structural grade, it probably is safe to consider it as meeting the next lower grade. It is important to understand that grading rules, themselves, are a function of the species and the grading association that promulgates the rules covering those species. What is select structural in one species and its associated rules, may not be select in another species. Furthermore, some species and grading associations do not have provisions for all grades for all species and in all timber size categories.

Once the species of the members in question and their corresponding grading are determied, then it is necesary to select an appropriate allowable stress. The NDS is the place to start, but the NDS-allowable stresses are based on extensive tests of newer wood samples and a fairly complex mathematical process is utilized to extrapolate raw laboratory tests of small clear specimens to allowable stresses in full-size members. One of the major components in deriving allowable stresses from laboratory tests is based on a probabilistic approach using a "95 percent exclusion rule." In simple terms, the allowable stress is based on a failure strength that 95 members out of 100 will exceed. In other words, only 5 members out of 100 will statistically possess a failure stress less than that used for determining the tabulated allowable stress. This particular aspect of timber design has no counterpart in other common structural materials, like steel or concrete, because of the variability of wood both among species and within the same species.

Some designers reasonably argue that, while this approach has served the design of new timber structures well for a long time, it may be too conservative for evaluating an historic timber structure such as a covered bridge. Those bridge components that have served successfully for at least fifty years (and maybe much longer) have been tested by time. Those components that contained the weaknesses implied by a 95 percent exclusion rule have either already failed or have not (yet) been highly stressed. Hence, it may be appropriate to consider some other exclusion limitation — some have suggested 80 percent. Yet, this issue has not been addressed by the current specifications and has not gathered much attention since it affects so few structures.

For more information on this topic, refer to guidance provided within the following American Society for Testing and Materials (ASTM) Specifications:

- ASTM D 143 — "Methods of Testing Small Clear Specimens of Timber
- ASTM D 198 — "Methods of Static Tests of Timbers in Structural Sizes"
- ASTM D 245 — "Standard Practice for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber"
- ASTM D 1990 — "Standard Practice for Establishing Allowable Properties for Visually-Graded Dimension Lumber from In-Grade Tests of Full-Size Specimens"
- ASTM D 2555 — "Standard Test Methods for Establishing Clear Wood Strength Values"
- ASTM D 2915 — "Standard Practice for Evaluating Allowable Properties for Grades of Structural Lumber"

So what is the modern analyst to do about the issue of allowable stresses in existing timber? At this writing, research continues occasionally aimed at this and related issues.

However, no definitive, published work exists that addresses this issue. This discussion is intended more to cause one to pause and consider the assumptions behind how historic covered bridges are evaluated. Until other information is available, it is prudent to continue to use the NDS-published allowable stresses as the basis for the analytical evaluation of historic covered bridge capacities.

Until the 1960s, the common practice was to establish the allowable tensile stress in wood members as a theoretical extrapolation of its measured failure stress in the tension zone of bending specimens. In part, this practice was followed due to the difficulties in developing tensile load testing equipment that could test timber specimens to failure without simply breaking at the grips. The grip zone failure casts legitimate doubts on the tensile stresses in the member at failure. Eventually, new grips were developed that led to satisfactory results in axial tension to failure tests. Unfortunately, the newer tests demonstrated that wood has less tensile strength than previously predicted by the flexural tests.

For smaller specimens, the differences between axially-induced and bending-induced tensile stresses were not major. However, as the size of the specimen increased, the differences became significant. The 1977 NDS introduced a new a reduction factor for allowable tensile stresses — up to 40 percent for members 10 inches (254 mm) and wider, and No. 1 grade or less. This new reduction obviously had a major effect on the allowable capacity calculations for timber trusses with larger chord and web members.

There are many practical examples of buildings that illustrate the rationale behind this change in the specifications. One notable example is the many World War II–vintage timber warehouse structures that were built with heavy timber trusses and that have demonstrated significant and frequent tensile fractures in the bottom (tension) chords.

Since evaluating timber covered bridge load capacities can lead one to consider changes in specifications over time — and the effects of such changes — it is good to keep this issue in mind. It also reinforces the importance of paying particular attention to the tensile members in covered bridge trusses.

Case Study — Rehabilitation of Burr Arch

Built in 1837, the existing Brown's River Covered Bridge, in Westford, Vermont, served until 1965 when it was bypassed and abandoned. Some repairs were performed in 1975. In 1987, once again in bad shape, the bridge was removed from its foundations and set on cribbing in a nearby field. Work on covered bridges was not as popular at that time as it is currently and the removal was performed with power provided by oxen and capstan. The effort to get the bridge off the foundations was filmed as part of a Public Broadcasting System documentary featuring Milton Graton (the late renowned covered bridge reconstruction specialist). Repair work was started by Graton, but funds were exhausted prior to the completion of the work.

The bridge sat idle in the field overlooking its foundations, awaiting additional funding to finish the project (see Figure 8). A long effort chasing grants culminated in funding via the Intermodal Surface Transportation Efficiency Act of 1991 and supplemental funding in 1999 from the Transportation Equity Act for the 21st Century.

With sufficient funds available, a contract was let for the final repair of the bridge and its abutments and for resiting the structure to its river crossing (see Figure 9). Bridge repair work commenced in early 2001 and it was relocated on July 20, 2001, via heavy moving steerable dolleys towed by large wrecker trucks. Back at its original site, adjacent to a modern bridge that supports vehicular traffic, the covered bridge now serves only pedestrian traffic.

Engineering & Construction Challenges. A number of issues caused this project to be especially challenging. From an engineering perspective, the lack of sufficient funds resulted in more reliance on the contractor to handle issues as they came up without thorough plans in advance. Multiple telephone conversations ensued during the work along with a few site visits. Fortunately, the contractor was

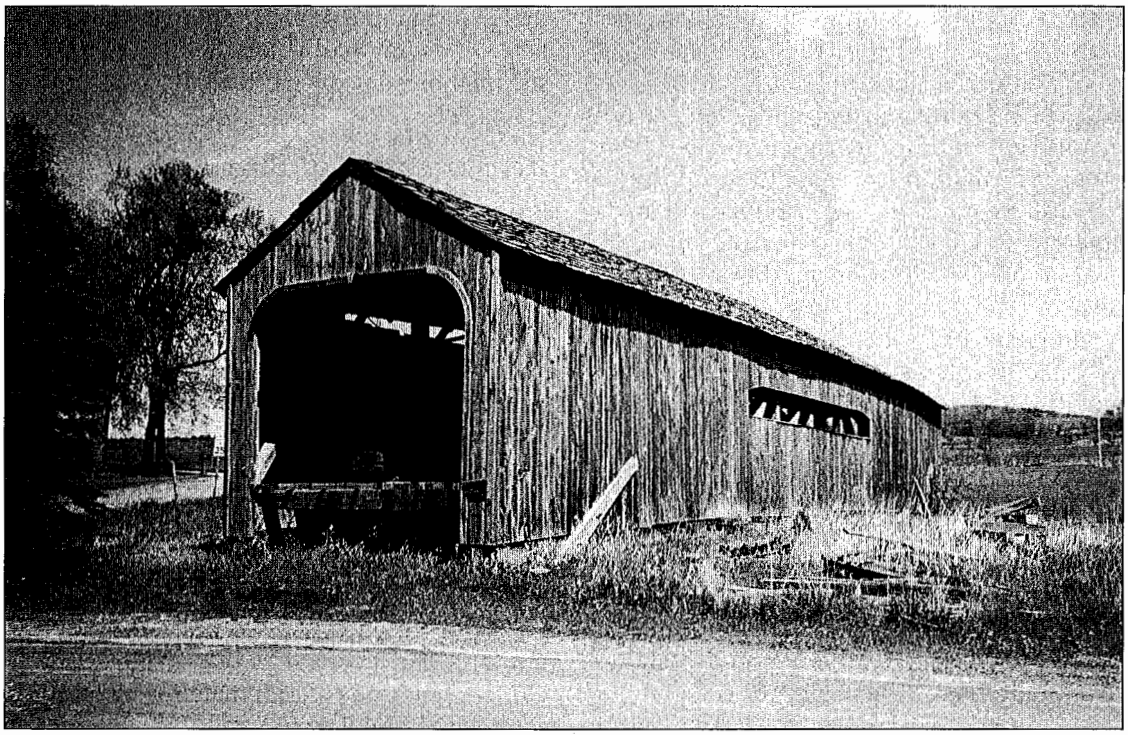

FIGURE 8. A photo of how the Brown's River Bridge sat for thirteen years off its original site.

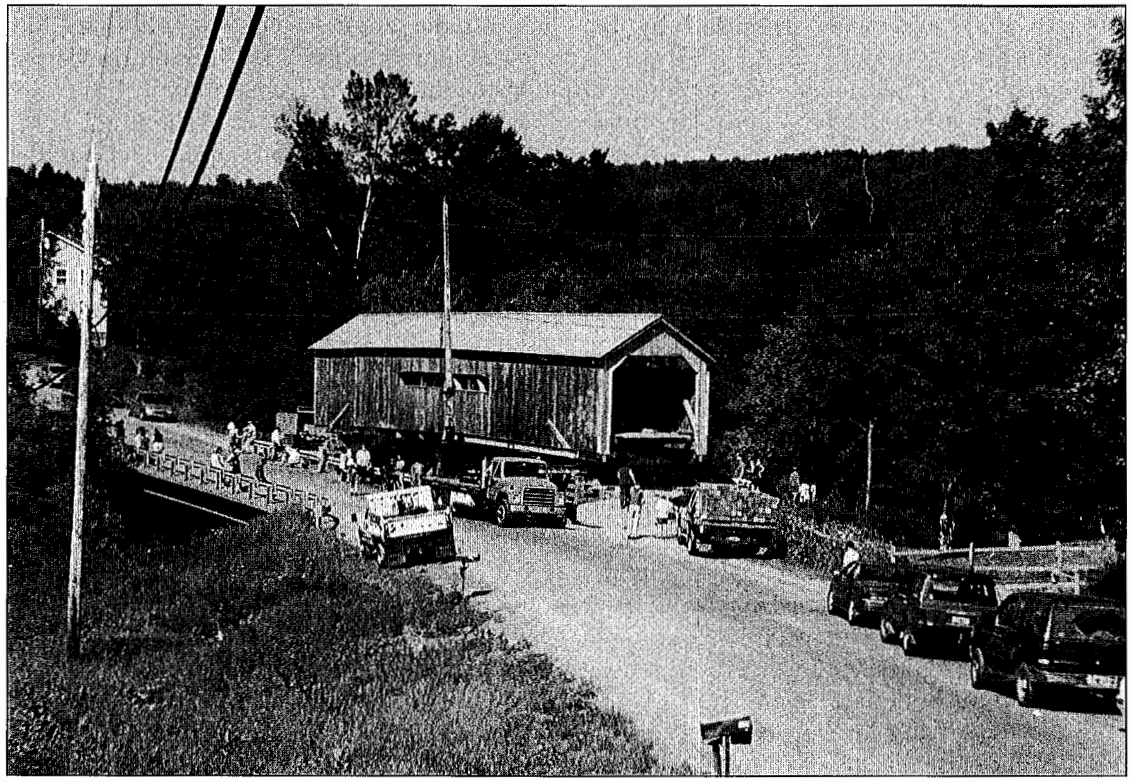

FIGURE 9. Moving the Brown's River Bridge to its new location.

especially conscientious and performed admirably under the circumstances.

Bridge Condition at the Start of the Project. Unfortunately, the repairs started in 1987 were not completed and the bridge sat on cribbing for thirteen years (see Figure 8). During that time, the cribbing suffered some differential settlement that racked the bridge. At the start of work, two-thirds of the top chord of the south truss had been attacked by powder post beetles (it cannot be determined if this damage was present before the 1980s). The wood shingle roof installed in the 1980s was leaking. The ends of the bottom chords had been replaced and large bolster beams were positioned beneath the chords. There was no direct splice of the chords and the connection of the bolsters was judged to be inadequate to restore the integrity of the bottom chords without additional work. Several damaged, broken or missing components of the bracing had not yet been addressed.

The abutments remained as they were when the bridge was removed. The upstream corner of the east abutment was separated from the stem and a section of stone wall along the wing wall was in near collapse condition. The caps and backwalls awaited replacement to accommodate the rebuilt superstructure.

Limited Engineering. At the time of the final push funds were extremely limited, yet the effort at obtaining the funds had been extraordinary (over thirteen years). Recognizing that the bridge would not have to support vehicular traffic and the pedestrian loading would be limited at this rural location, an engineering analysis of the bridge included:

- Evaluating the current condition of the bridge;
- Identifying necessary repairs based on judgement and experience with covered bridge work;
- Conducting a limited analysis of critical elements, as judged mandatory (primarily limited to the connection of the bolster beams to the bottom chord);
- Preparing limited drawing details;
- Preparing technical specifications with reliance on lump-sum bid items for the several major tasks, including timber structure repairs, foundation repairs, timber structure relocation and timber flooring; and,
- Scheduling site attendance at the time of the relocation of the bridge to evaluate the proposed work plan and equipment.

It should be noted that another local engineering firm was retained by the Westford Historical Society to prepare a complete contract bid document, including bid forms, agreement, bond requirements and other standard information. While this situation is not desirable, and should not be undertaken by inexperienced personnel, this project approach proved to be successful in this instance, in large part due to the close working professional relationship among the contractor, engineer and owner's representative.

Timber Repairs. One of the important issues related to this work was the need to replace truss components while attempting minimal removals. The top chord of the east truss was originally built in three pieces with a special joinery connection (see Figure 10). The chords were positioned down onto tenons from the ends of the posts. The replacement of the chord therefore required the temporary support of the rafters and tie beams to raise them out of the way.

While replacing the top chord, a section of substantial rot was found in the top of one post (see Figure 11). No external evidence of such deterioration was visible. Yet, the post either had to be partially or completely replaced. The decision was made to completely replace the post. A couple of other posts were also replaced to better address previous partial repairs that were judged to be insufficiently connected.

Straightening the structure as a consequence of these rather extensive repairs and long-term cribbing differential settlement led to discovery that the existing knee braces would have to be replaced to fit the restored geometry.

One of the features often added by Graton in his restoration of scores of covered bridges were bolster beams beneath the ends of the bottom chords. These elements project beyond

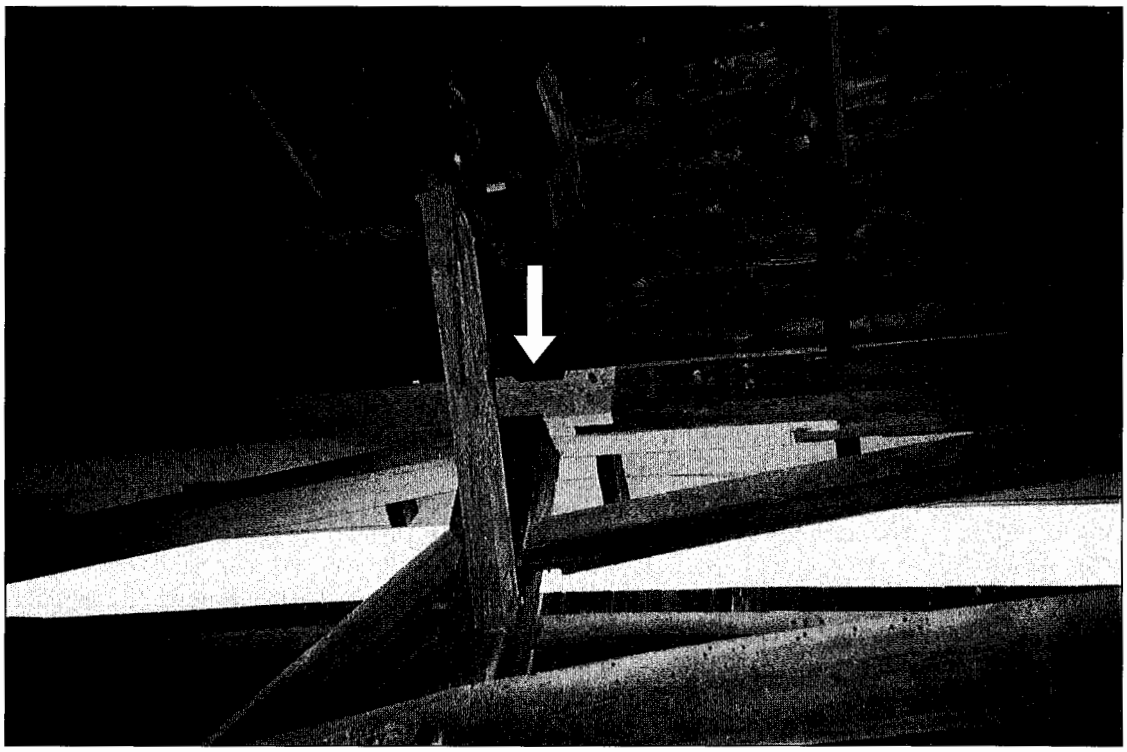

FIGURE 10. Replacement top chords for the Brown's River Bridge.

the front edge of the abutment seat and serve to stiffen the ends of the truss. However, the bolsters added to this bridge were especially large and long — in fact three panels long (see Figure 12). Bolsters are usually added separately from the chord so that they do not act as truss members — instead, they supplement the behavior of the truss by adding bending capacity in addition to the truss behavior. The existing bottom chords had been severed at sound material away from the rotted end portions, in the second or third panel from the end. Instead of installing a tensile splice to connect the new ends to the original portions, the large bolsters were connected to the chords by a few vertical bolts. These connections were not completed at the time work was abandoned.

However, the bolsters were expected to serve as replacement bottom chord elements being subject to axial loads in addition to their normal bending behavior. While the bolsters were no doubt large enough to accommodate such expectations (4.5 by 16 inches [114 by 406 mm]), they not been connected in a way to adequately transfer the loads across the butt connections of the bottom chord.

Accordingly, a plan was devised to install a series of timber shear blocks at the interface of the bottom of the chord to the top of the bolster. In combination with the bolts, the additional shear blocks would provide an acceptable capacity. The plans called for rectangular 4-inch (100-mm) high by 8-inch (200-mm) wide hardwood blocks to be inserted into new pockets cut into the interface from the side. The contractor had considerable experience with use of round wooden dowels to accomplish the same effect and believed the round dowels would serve better in this instance. Following extensive conversations and additional analytical evaluations, round dowels were used (see Figure 13).

Modified Burr Arch Details. In Figure 12 it can be seen that the arch elements have been cut off at the bottom of the bolster beams. In a classic Burr Arch, the arch elements project down past the bottom of the chords to bear against the abutment. In so doing, they can true arch behavior to augment the truss

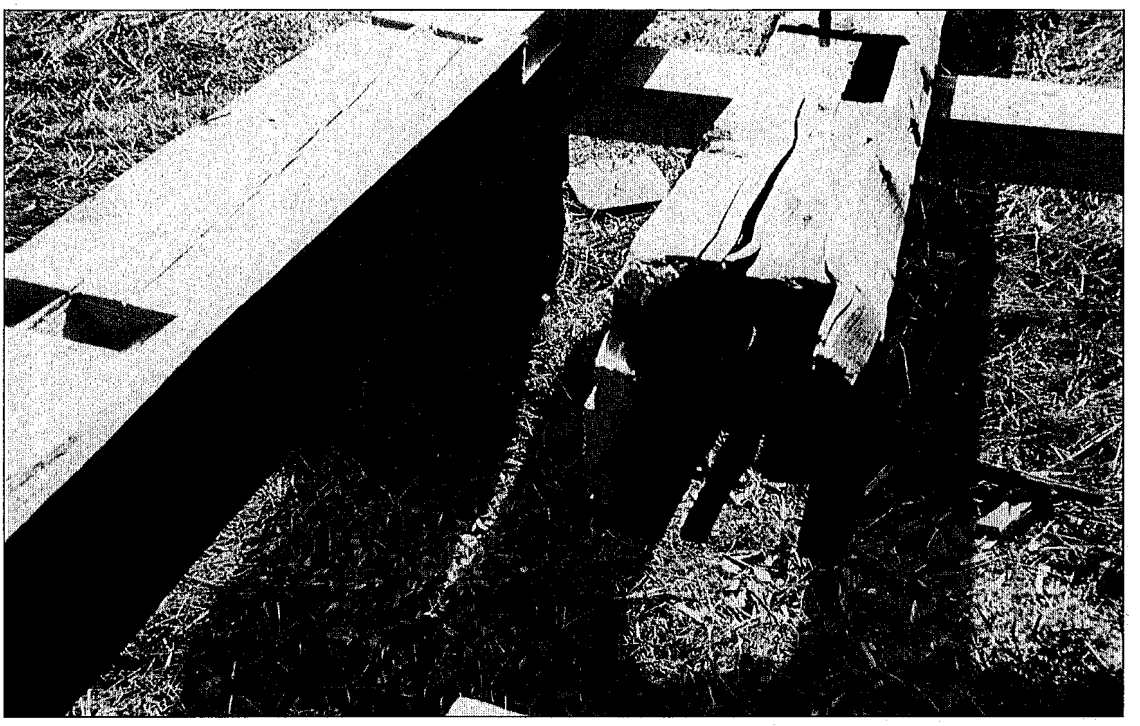

FIGURE 11. Hidden rot inside the top of an old post on the Brown's River Bridge.

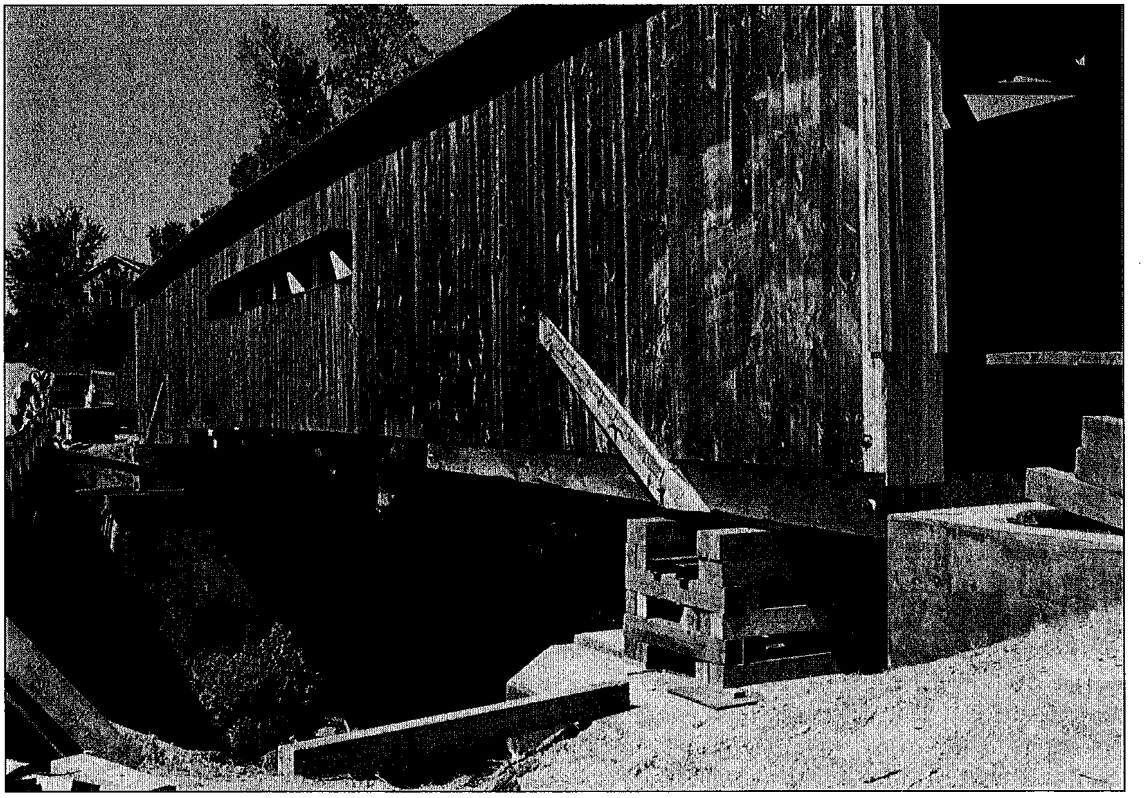

FIGURE 12. The Brown's River Bridge was moved but not down off its temporary cribs.

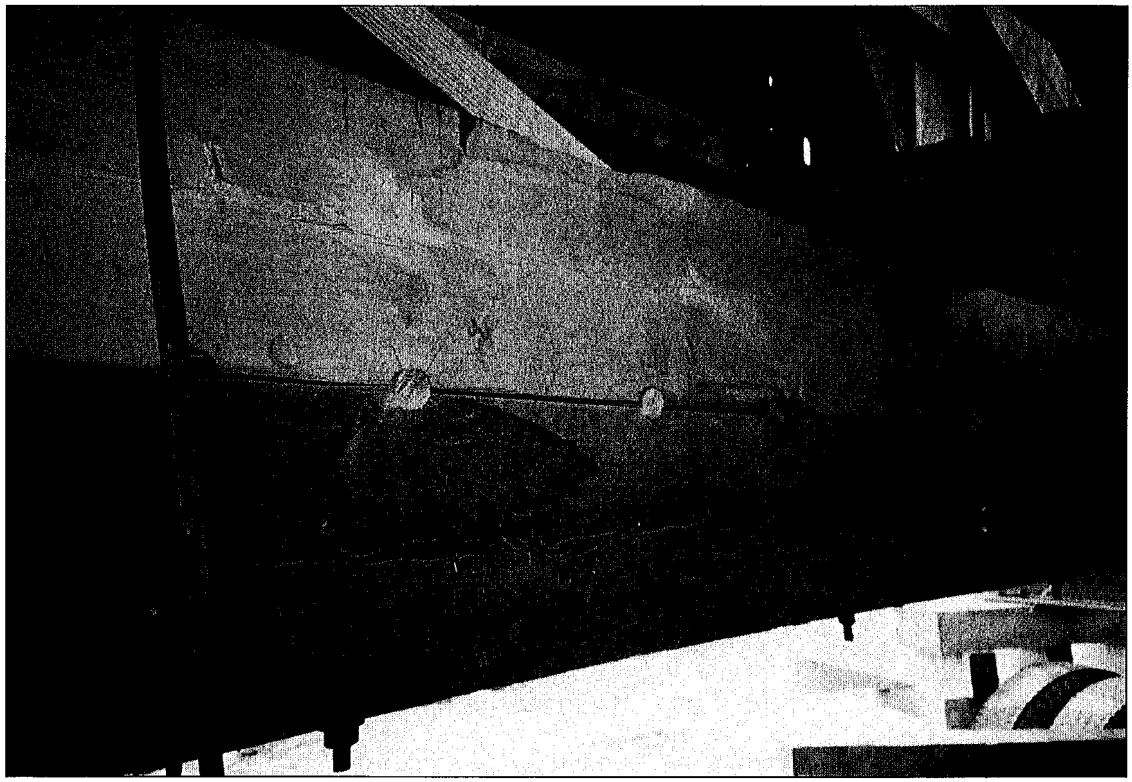

FIGURE 13. A view of shear dowel reinforcement on the Brown's River Bridge.

behavior of the timber structure. However, in those bridges that have had an alteration to terminate the arch without bearing on the abutment, or if the bridge had been built originally like that, the arch cannot offer much assistance to the truss. Such structures are often times referred to as "modified" Burr Arches.

Furthermore, the connection between arch and truss is often a single bolt element at the intersection of the center of the vertical elements of the truss and the centerline of the arch. It is common to observe that these bolt elements are deformed from the excessive forces attempting to connect the two types of systems together, which is another reason that makes the analysis of such combination structures especially challenging.

The arch components on this bridge are comprised of a pair of solid sawn members that straddle the truss. As the arch in the center of the bridge comes lower towards the bottom chord as it traverses toward the end of the bridge, it has been deflected outward in this bridge to pass the bottom chord. Hence the gradual appearance of the arch in Figure 12. Some bridges are built so that the arch components are interrupted at the bottom chord with a splice in order to ensure that the arch remains in the same plain as the bottom chord.

Foundation Considerations. The original stone abutments had been faced with concrete (a common practice). While the quality of concrete facing was not great, the bulk of it seemed sound and stable. There was no evidence of deep-seated foundation failures; hence it was quickly decided to retain the bulk of the abutments. New caps were required due to a change in the floor configuration. (The trusses originally extended beyond the floor; therefore, the concrete backwalls had jogs at each corner.) The north corner of the east abutment also had separated from the main stem concrete and needed to be replaced. The connecting wing wall was dry laid stone that had shifted and was on the verge of collapse. A roadway drainage pipe

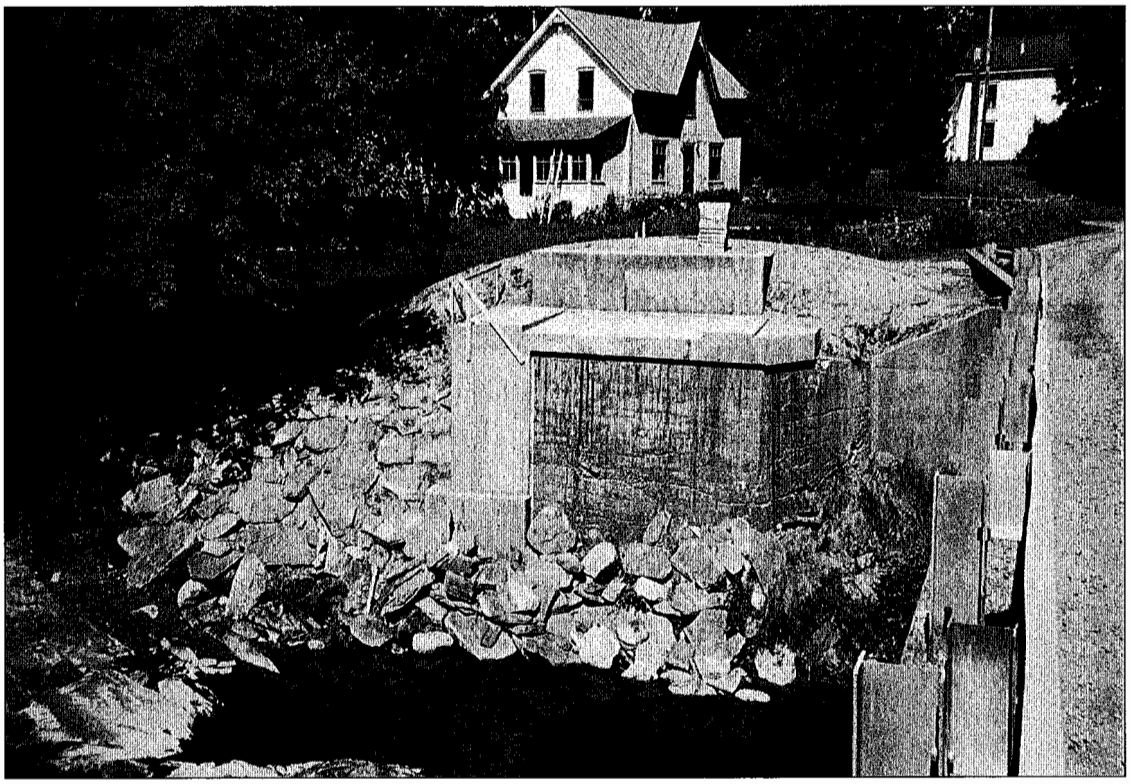

FIGURE 14. A view of the reconstructed east abutment for the Brown's River Bridge.

discharged at a point at the beginning of the stone wall and was contributing to its demise. A review of the roadway drainage situation indicated that a cross culvert could be installed in the roadway uphill of the covered bridge abutment. Doing so would permit removing a major portion of the stone wingwall and regrading in order to fill in the ditch in front of the wall. The corner of the abutment was removed and recast on new concrete stepped footings to connect to the rest of the abutment stem concrete.

New thick concrete pads were cast atop both abutments to help knit the combination of original stone and concrete facing together. Backwalls and short wing walls were then cast with dowels into the main pads. The resulting concrete treatment was economical and should serve well for a long time (see Figure 14).

Roof Replacement. The bridge had had new wooden shakes installed in the 1980s, but the roof was leaking quite badly. After consideration of the alternatives, it was decided that the roof should be replaced. Vehicular traffic tends to vibrate a bridge enough to cause snow to slide off faster than bridges closed to traffic. Since this bridge was to remain only open to pedestrian traffic, it seemed prudent to use metal panels rather than wooden shakes or shingles.

Relocating the Bridge. Without question, the relocation of the bridge garnered the most attention. The general contractor decided to hire the services of a company that specialized in moving structures. A pair of steerable dolleys was located at each end of the bridge. The power to move the bridge was provided by winches from large wreckers. The roadway is quite steep leading down to the bridge site and the process involved parking a tractor uphill of the bridge and releasing the cables to the limit of the cable spool, then moving the wrecker to start the process again. The far end of the bridge was hooked to a separate wrecker that provided the steering guidance. It was able to hold the bridge in position with its brakes while the rear wrecker was relocated

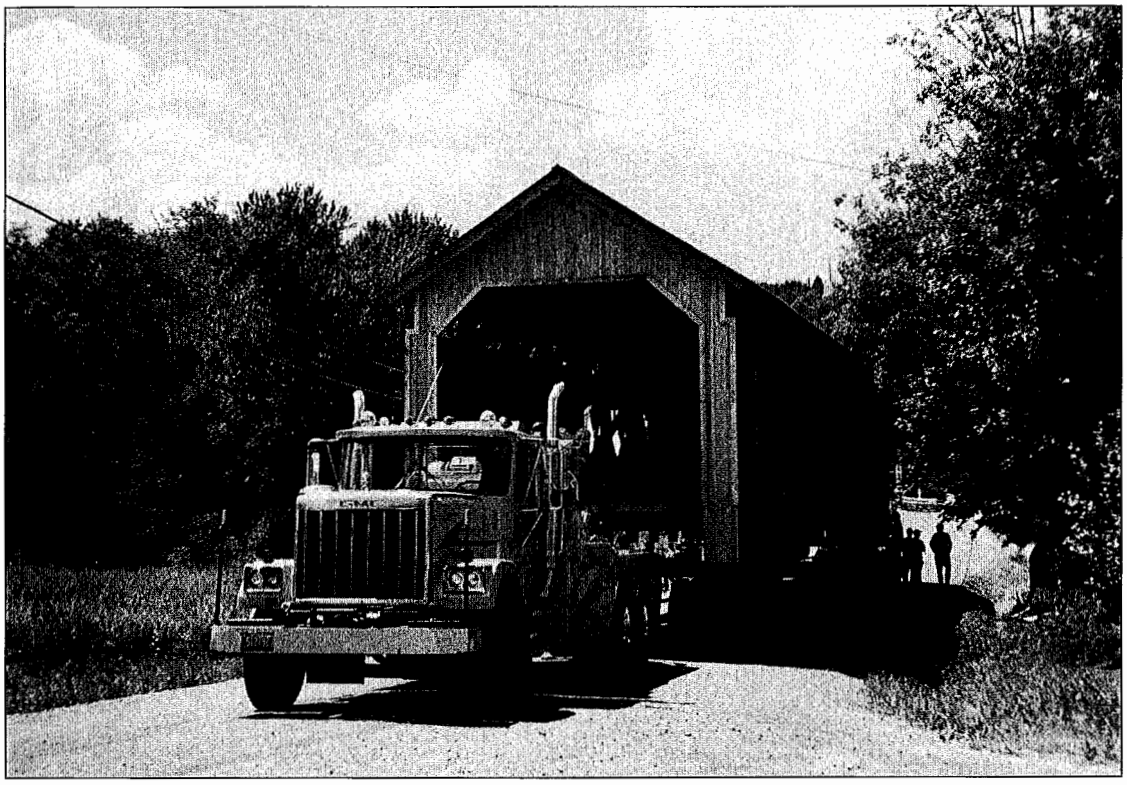

FIGURE 15. The Brown's River Covered Bridge underway on dolleys.

(see Figure 15). Upon reaching the low point in the profile, the far wrecker took over and pulled it into place.

The close proximity of the bypass bridge was very convenient in that the covered bridge on the steerable dollies was pulled across the bypass bridge to align it on the abutments. Then cribbing and skid beams were installed beneath the bridge superstructure to allow it to be winched sideways over the top of the guiderail to the rehabilitated abutments. An interesting challenge involved the relative skew between the two bridges (they are not parallel). Hence, the end of the bridge closest to the old location was supported on a single heavy roller nest at the centerline of the bridge to allow it to act as a pivot when the other end was winched sideways. In this way, the bridge was moved differentially to align it with its bearing areas. The movers used a swivel roller system beneath each corner at first. Then to rotate the bridge to accommodate the skew between the respective bridges, the pair of rollers was replaced with a central unit at one end to provide a pivot point (see Figures 16 and 17).

Once in the proper location, the bridge was too high and had to be lowered into place by a repetitious process of supporting the weight on the jacks and then removing the cribbing, then releasing the jacks to the new lower position. The move from the field to its original location over the river was accomplished by mid-afternoon. The lowering process was not completed until the following morning.

Case Study — Rehabilitation of a Long Truss

The Hamden Covered Bridge, built in 1859, spans the West Branch of the Delaware River near the hamlet of Hamden in Delaware County, New York (see Figure 18). The county, as owner, undertook a major rehabilitation of the bridge in 2000. The bridge is supported by truss configurations patented in 1830 by, and named for, Colonel Stephen H. Long.

The contractor chose to relocate the bridge to allow work on land. Unfortunately, the relo-

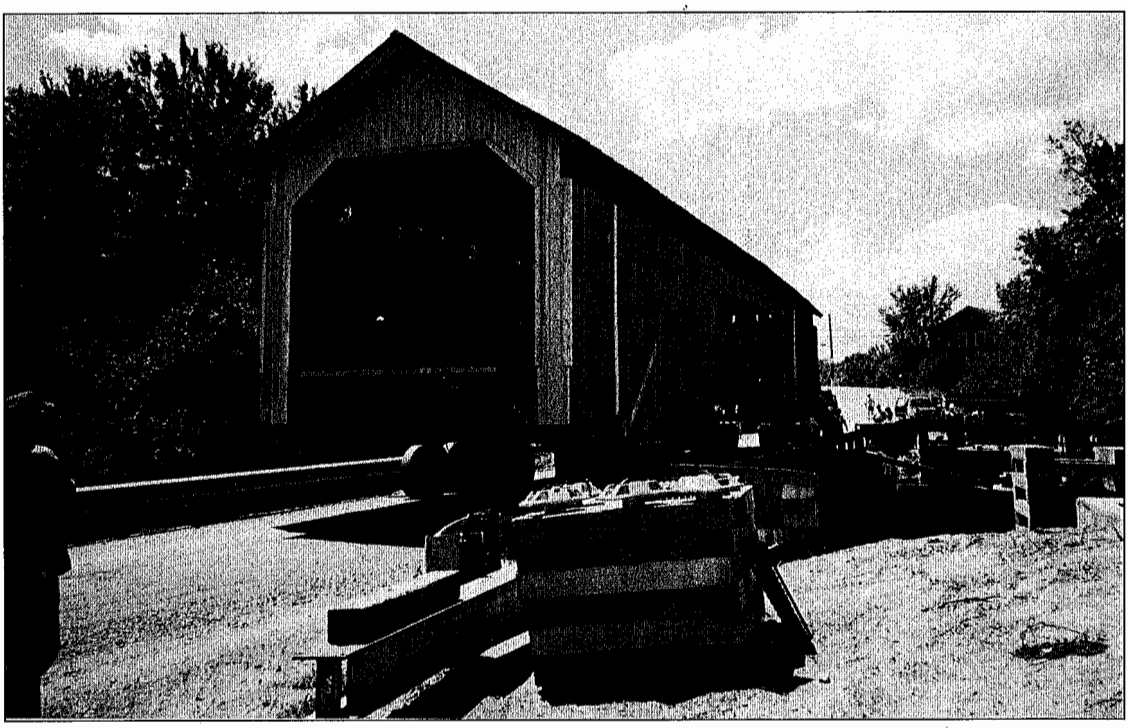

FIGURE 16. Atop the bypass bridge and ready to be slid sideways.

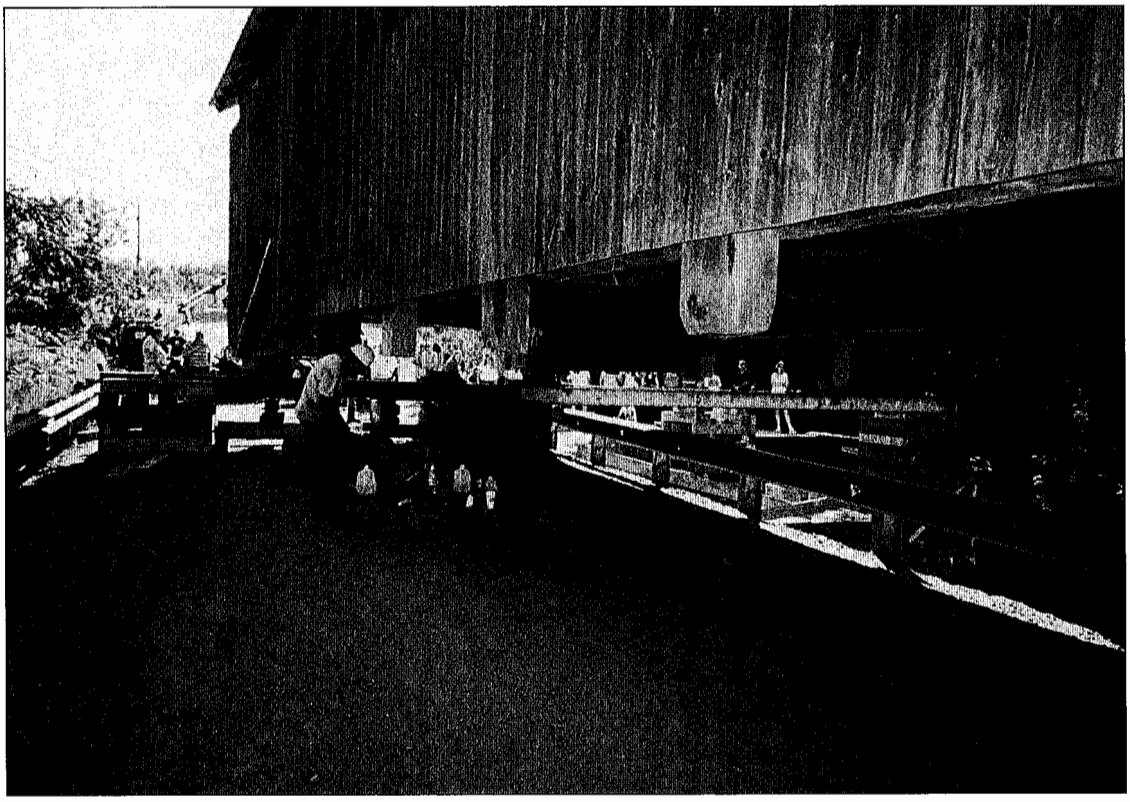

FIGURE 17. The Brown's River Covered Bridge moving sideways.

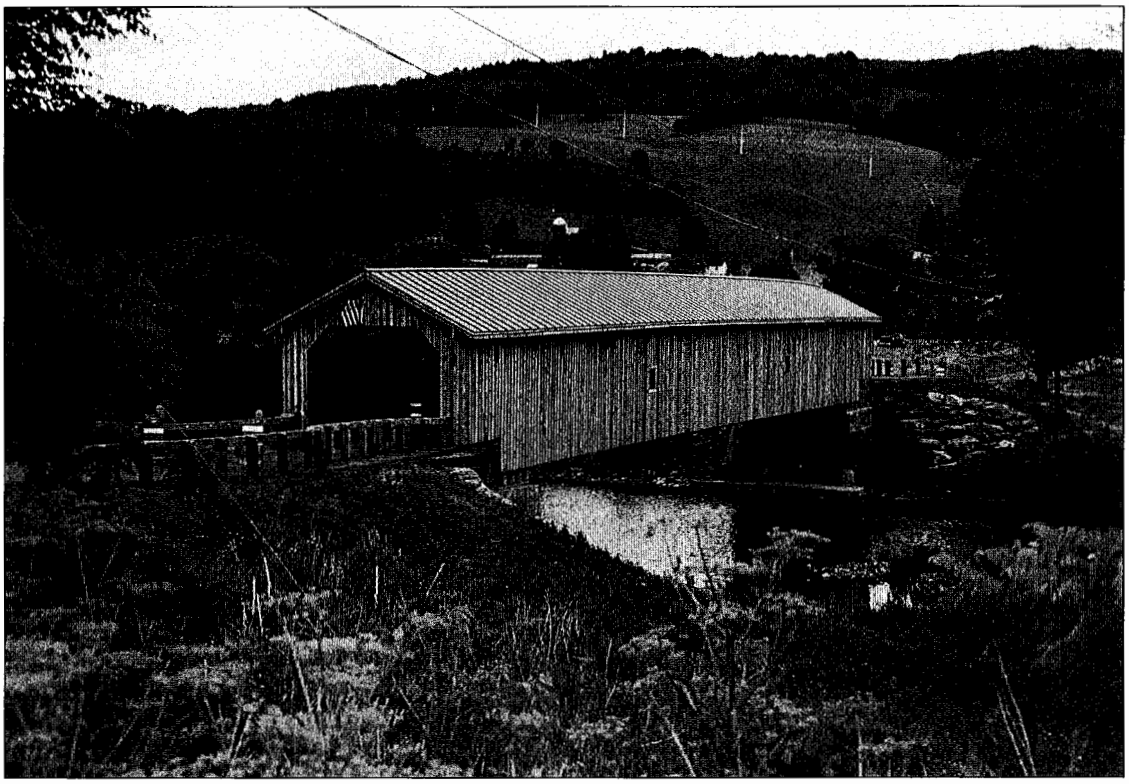

FIGURE 18. A view of the Hamden Covered Bridge.

cation procedure using a pair of cranes experienced an accident that culminated in breaking the single-span bridge into two parts. Careful retrieval of the damaged trusses and patience and diligence succeeded in the rehabilitation and resiting of the bridge.

The style of truss used on this bridge contained three separate bottom chord sticks with a pair of vertical posts between them. The chord members contained a larger central stick of 9 by 13 inches (229 by 330 mm), flanked by smaller sticks of 5 by 13 inches (12 by 330 mm) with lengths of up to 50 feet (15.2 m). The traditional form of connection for such an arrangement at the time of its construction uses a splice termed a "bolt-of-lightning" detail. Unfortunately, the proportions of the truss and its joinery was such that the splice was substantially weaker than the main members. Based on experience with similar circumstances encountered earlier at another county-owned covered bridge, it was decided to replace the bottom chords with three one-piece glue-laminated members.

Other problems encountered in this project included those with bracing, counter members, connectors in joints of Long trusses and the myth and reality of Colonel Long's wedges.

The work was completed in June 2001 for a total cost of approximately $750,000.

Case Study — Rehabilitation of a Town Lattice

Fitch's Covered Bridge was originally built in 1870 on Kingston Street over the West Branch of the Delaware River in the Village of Delhi, Delaware County, New York (see Figure 19). The bridge is supported by Town lattice trusses, so named for Ithiel Town who received his first patent for this truss configuration in 1820.

Several problems requiring solutions were encountered in this bridge rehabilitation, including:

- Hidden deterioration (a common problems with lattice trusses);
- Figuring out how to restore the trusses to their original configuration;

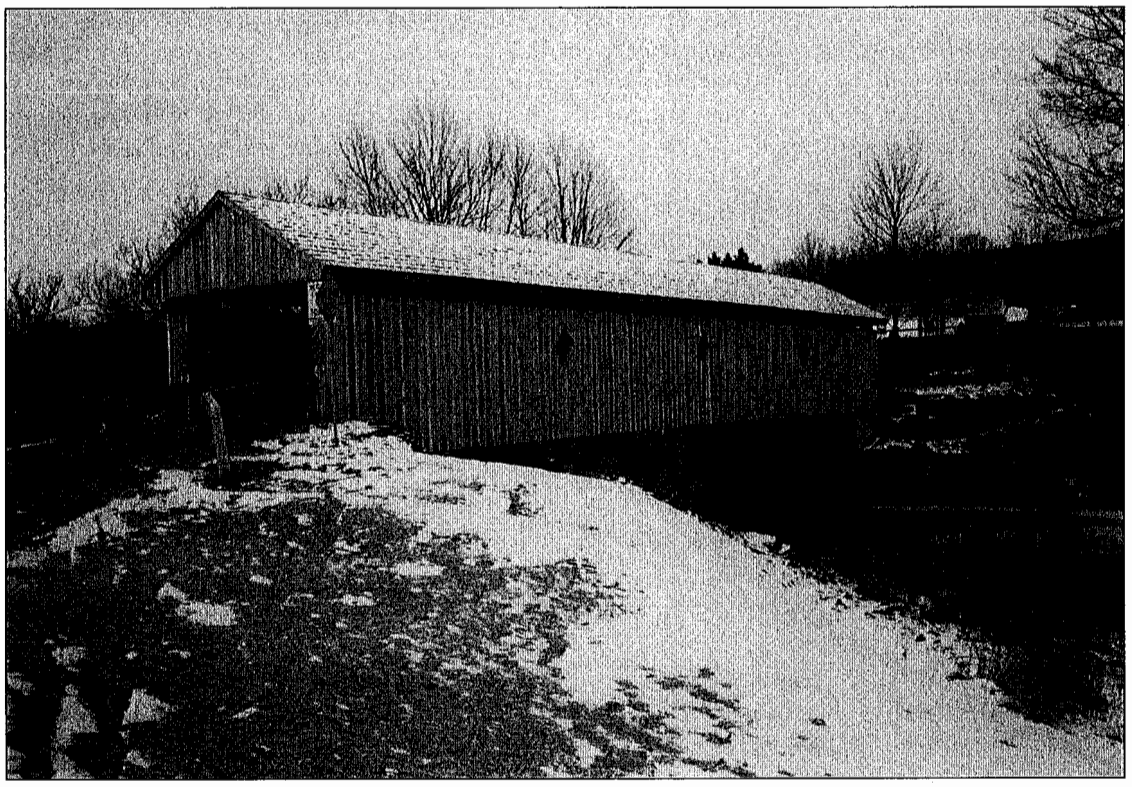

FIGURE 19. A view of Fitch's Covered Bridge.

- Dealing with the vertical camber of the tusses (how to deal with existing sagged structures);
- Insufficient bracing;
- A weak floor system; and,
- Devising a way to retain the existing foundations.

Field instrumentation of portions of the existing Town lattice trusses was performed to assess the distribution of forces around chord interruptions and trunnel (wooden peg) connections of chords and lattice elements. This work was apparently the first of its type for this kind of application.

All design and construction work for the rehabilitation of the Fitch's Covered Bridge was performed by county forces. The cost of the work was about $425,000.

ACKNOWLEGMENTS — *Portions of this article were adapted from material prepared by a team led by Phillip C. Pierce that will be soon published as the* Covered Bridge Manual *by the Federal Highway Administration. This article was also abridged from a longer paper presented to, and published, in the BSCES Structural Group proceedings of 2003. The 1992 Vermont Agency of Transportation study was performed by a consulting team led by the firm of McFarland-Johnson from Binghamton, New York. The author served as the Project Manager for that challenging and intriguing project. The contractor for the Brown's River Bridge restoration was Renaud Bros., Inc., in Vernon, Vermont. The author performed the engineering analysis of the Brown's River Bridge. Lamoureux and Dickenson Consulting Engineers, Inc., Essex Junction, Vermont, prepared a complete contract bid document for the Brown's River Bridge. Caroline Brown, President of the Westford Historical Society, was the representative of the society for the rehabilitation of the Brown's River Bridge. Messier House Moving, East Montpelier, Vermont, was hired to move the rehabilitated Brown's River Bridge. The section on the rehabilitation of the Hamden Covered Bridge, Delaware County, New York, was adapted from an article by the author that was published in the* Transporta-

tion Research Record *No. 1792. The section on Fitch's Covered Bridge was also based on an article published in the August 2001 edition (Vol. 6, No. 3) of ASCE's* Practice Periodical on Structural Design and Construction.

PHILLIP C. PIERCE *received his B.S. in Civil Engineering in 1972 and his M.S. in 1983 from Pennsylvania State University. He has over thirty-two years of diverse and extensive experience with all forms of bridges, in addition to work on various forms of buildings, other structures and general civil engineering projects. Having worked for several consulting engineering firms, he now serves as Deputy Commissioner in charge of engineering at the Delaware County (New York) Department of Public Works, where he is responsible for over 400 major highway structures and 270 miles of road. He also operates a specialty consulting engineering practice with major focus on covered bridges. He recently completed preparation of the* Covered Bridge Manual, *which will be published by the Federal Highway Administration.*

REFERENCES

1. The National Society for the Preservation of Covered Bridges, *World Guide to Covered Bridges*, 1989.

2. Graton, M.S., *The Last of the Covered Bridge Builders*, Clifford-Nichol: Plymouth, New Hampshire, 1978.

3. Advisory Council on Historic Preservation, *http://www.achp.gov/secstnd.html.*