Structural Modeling for Lateral Stiffness in Historic Truss Bridges

FREDERICK R. RUTZ, KEVIN L. RENS, VERONICA R. JACOBSON, SHOHREH HAMEDIAN, KAZWAN M. ELIAS, and WILLIAM B. SWIGERT

Inclusion of stiffening elements, such as decks, into structural-analysis models can aid engineers in preserving historic bridges.

Introduction

Historic truss bridges from the late-nineteenth and early-twentieth centuries are vanishing rapidly; it is estimated that half of the nation's truss bridges that existed just 20 years ago have since been removed.[1] At this rate of attrition, the engineering legacy of the truss bridge may soon be relegated to the history books.

One avenue for preservation is rehabilitating such bridges for pedestrian use. While the principles espoused in this paper are equally applicable to bridges still intended for vehicular traffic, rehabilitation of former highway bridges to pedestrian use is the focus of this study. Conversion of truss bridges to pedestrian use permits ready public access to historic structures and provides incentives for the bridges' continued maintenance.

Fig. 1. Fruita Bridge over the Colorado River, near Fruita, Colorado. This Parker truss, built in 1907, has three 155-foot (47-meter) spans, each with eight bays. Its deck consists of timber deck planks spiked to timber stringers, which bear on steel floor beams. The deck is discontinuous, with gaps between the deck planks. All images by the authors.

Unfortunately, the engineer involved with a historic-bridge preservation project often finds that the bridge has insufficient lateral strength to satisfy modern requirements.[2] This perceived deficiency is due to two related circumstances: the present-day requirement for wind load is significantly higher than that used for the original design a century ago, and the use of traditional structural analysis can lead to an incorrect conclusion that the wind load will result in structure overstress, causing the bridge to fail. The issue of wind pressure required in modern design is not addressed here. Rather than engaging in often-futile argument with code officials over the allowance for wind pressure mandated for design, a methodology for addressing the second circumstance by utilizing modern structural-analysis tools is presented.

Goals

Research on this topic at the University of Colorado at Denver has focused on the stiffening effect of decks in historic truss bridges. There is strong evidence that decks stiffen a bridge both vertically and laterally, although traditional analysis methods, limited to structural skeletons only, typically ignore this influence. Accounting for the stiffening effect of a bridge deck is analogous to including such effects from floors, interior partitions, and roofs in buildings. While the overall purpose of this more-comprehensive approach is to aid in preservation efforts for historic iron- and steel-truss bridges, the specific goal of this project is to demonstrate a new methodology to account for lateral strength provided by nontraditional (but real) load paths. Although the focus of this study is sur-

Table 1. Summary of Wind-Pressure Recommendations for Bridges from the Late-Nineteenth and Early-Twentieth Centuries

Source	Span 60 ft. (20m)	100 ft. (30m)	200 ft. (60m)	1,000 ft. (300m)	1,500 ft. (450m)
C. Shaler Smith, "Wind Pressure Upon Bridges," *Engineering News* (Oct. 1, 1881): 395.	30 psf (1.44 kPa)	30 psf (1.44 kPa)	30 psf (1.44 kPa)	30 psf (1.44 kPa)	_
J. A. L. Waddell, *The Designing of Ordinary Iron Highway Bridges* (New York: John Wiley & Sons, 1884), 6.	_	40 psf (1.92 kPa)	35 psf (1.68 kPa)	30 psf (1.44 kPa)	30 psf (1.44 kPa)
J. A. L. Waddell, *De Pontibus* (New York: John Wiley & Sons, 1898), 224 and Plate VIII.	40 psf (1.92 kPa)	40 psf (1.92 kPa)	_	_	25 psf (1.20 kPa)
Theodore Cooper, "What Wind Pressure Should be Assumed in the Design of Long Bridge Spans?" *Engineering News* (Jan. 5, 1905): 15-16.	50 psf (2.40 kPa)	30 psf (1.44 kPa)	30 psf (1.44 kPa)	_	_
J. A. L. Waddell, *Bridge Engineering* (New York: John Wiley & Sons, 1916), 149-154.	35 psf (1.68 kPa)	35 psf (1.68 kPa)	35 psf (1.68 kPa)	25 psf (1.20 kPa)	_

viving metal-truss bridges, the principles hold true for timber trusses as well.

Rehabilitation for Preservation

Rehabilitation of truss bridges for pedestrian use is a practical and popular way to preserve these historic structures. However, the American Association of State Highway and Transportation Officials' (AASHTO) *Guide Specifications for the Design of Pedestrian Bridges* mandates a relatively stringent wind-load design criteria.[3] Structural engineers attempting to rehabilitate historic bridges from highway to pedestrian use often discover that the old structures lack the strength to resist the AASHTO wind-load criteria. This apparent inadequacy can lead either to a draconian structural retrofit, which is both expensive and detrimental to the historic character to be preserved in the first place, or to condemnation of the bridge. Although traditional structural analysis may deem many historic bridges inadequate, in case after case observations reveal no physical evidence to suggest that wind has caused damage or distress, even after a century of exposure.[4] At this age, bridges have weathered many severe windstorms. Thus, the evidence suggests these structures have better resistance to wind pressure than what is revealed by traditional analysis alone.

Modeling and Analysis Background

Traditional structural analyses of truss bridges are based on a skeleton-frame analysis, the classic textbook method, which has been used since Squire Whipple published the method of joints in 1847.[5] The "computer" in the nineteenth-century design office was the individual who performed the calculations, using the classic methods of joints and sections or perhaps graphical methods that simplified some of the arithmetic, to determine bridge member forces. Today's practitioner using one of the many readily available computer programs is really utilizing matrix algebra. The computer is now a machine, but it does the same job the human "computer" once did — it completes the calculations. While the techniques of analysis have changed from hand calculations to computer analysis, the basis for analytical models has remained basically unchanged.

There are many instances of the results of gravity-load tests demonstrating that the vertical stiffness is actually greater than that calculated by skeleton analyses.[6] While there are examples (such as the Cornish-Windsor Covered Bridge, which spans the Connecticut River between Vermont and New Hampshire) of engineers having included the deck in the lateral analysis, little information on this method can be found in the literature.[7] This paper is offered to help fill a void in the literature by describing a method the authors used to evaluate five different metal-truss bridges.

Loads

Superimposed dead load and superimposed live load are still computed manually in the same way that was used by the nineteenth-century designer. Self-weight may be computed manually or determined by software. The AASHTO *Guide Specifications for the Design of Pedestrian Bridges* prescribes the live-load and wind-load values. Current live loads may vary from 65 psf to 85 psf (3.11 kPa to 4.07 kPa), depending on the area of the walkway.

In the absence of standards, nineteenth-century bridge engineers based the determination of wind loads on their own reasoning (Table 1). The recommendations of C. Shaler Smith, J. A. L. Waddell, and Theodore Cooper appear to be based on their own conclusions, formed from years of experience in bridge engineering. While the thinking of all bridge engineers probably evolved over this period, that of Waddell is well documented in his three books, which provide insight into the reasoning behind bridge-design criteria in his day.[8] Note that for spans in the range of 100 to 200 feet (30 to 60 m), typical for many surviving truss bridges today, the design

Fig. 2. Blue River Bridge over the Blue River, near Silverthorne and Dillon, Colorado. This steel Pratt truss has five bays and an 80-foot (24-meter) span. It is believed to have been built in about 1895 as Two-Mile Bridge near Breckenridge, Colorado, and moved to this site at a later, but unknown, date.

Fig. 3. Prowers Bridge over the Arkansas River, near Lamar, Colorado. The span selected for study is a nine-bay camelback Pratt through truss of 160-foot (49-meter) span, built in 1909. It has steel floor beams with steel stringers, covered by a corrugated-metal deck with asphalt pavement.

wind pressure was on the order of 30 to 40 psf (1.44 to 1.92 kPa). This is significantly lower than today's AASHTO *Guide Specifications for the Design of Pedestrian Bridges*, which mandates 75 psf (3.59 kPa), about double the original design value. It should be noted that typical allowable stresses for iron and steel are presently at higher levels than when the bridges were designed.[9]

The Five Bridges Studied

Five historic truss bridges located in Colorado were studied: Fruita Bridge in Mesa County, Blue River Bridge in Summit County, Prowers Bridge in Bent County, Rifle Bridge in Garfield County, and San Miguel Bridge in Montrose County (Figs. 1 through 5). The five bridges had a number of features in common:

- pin-connected trusses
- through trusses
- Pratt trusses or Pratt-truss derivatives
- late-nineteenth- or early-twentieth-century construction
- metal, either wrought iron or steel
- abandonment (except one)
- geographic region

Other features varied among the bridges:

- spans
- railings
- varying degrees of deterioration and damage
- deck construction (an important aspect of this study)

Methodology Overview

In this study, all five bridges were analyzed by the traditional skeleton approach using 3-D structural-analysis software typical of the software tools commonly utilized by practicing engineers. All analysis was performed with either RISA-3D or RAM Advanse, software which includes both frame elements and plate/shell elements and is readily available to practicing engineers.[10] Other software with similar capabilities is also available.

As-built dimensions and section properties were used, making this a study of real-world bridges, not a theoretical ex-

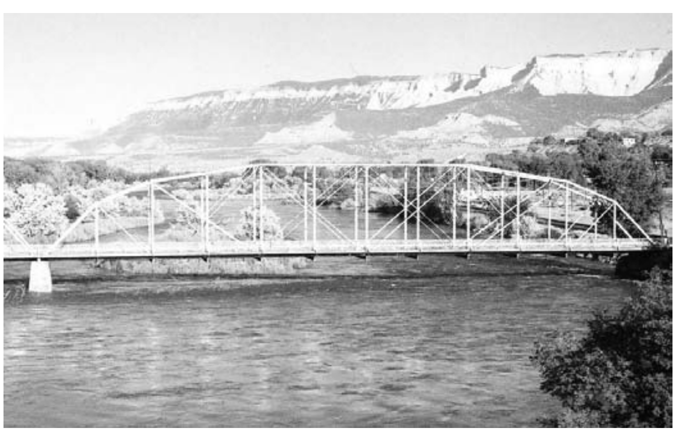

Fig. 4. Rifle Bridge over the Colorado River at Rifle, Colorado. With a 240-foot (73-meter) span, this steel Pennsylvania truss, built in 1909, comprises the longer of two different spans at that location. The deck is similar to that at Prowers: steel floor beams with steel stringers, covered by a corrugated-metal deck with asphalt pavement.

Fig. 5. San Miguel Bridge over San Miguel River in western Montrose County, Colorado. This wrought-iron bridge with a 142-foot (43-meter) span was built in 1886. In 1964 its timber deck and stringers were replaced with steel stringers with semicircular segments of corrugated-metal pipe that bear on the bottom flanges of the stringers, topped with gravel roadbed.

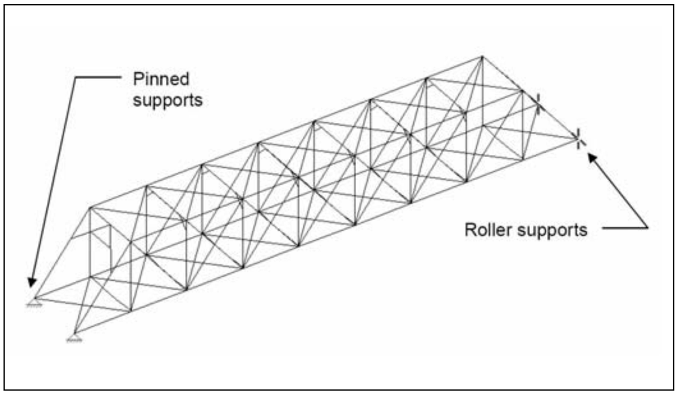

Fig. 6. A typical 3-D skeleton model, illustrating the traditional skeleton based on the steel members only. Frame elements were used for all members. The boundary conditions — pinned at one end and rollers, restrained from translation in the lateral direction, at the other end — are indicated. This model is of San Miguel Bridge.

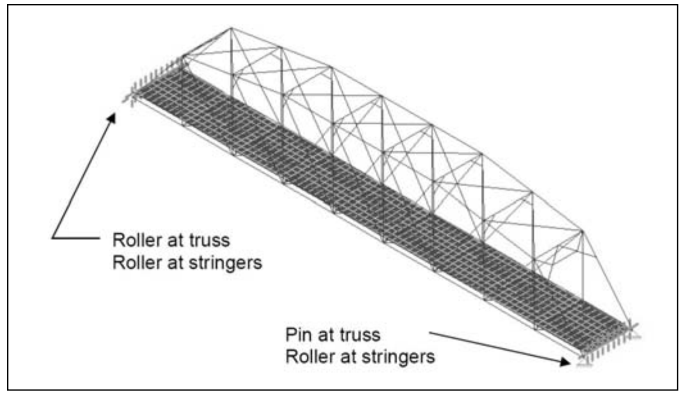

Fig. 7. A typical 3-D deck model. The deck is modeled as plate elements, added to the skeleton model. The stringers were added to the skeleton model as frame elements, and the deck was added using plate elements. This model is of Prowers Bridge, which has the plate elements interconnected at all their nodes, or corners.

ercise. For all five bridges studied, skeleton-frame models were used, based on:

- AASHTO wind load determined from a pressure of 75 psf (3.59 kPa).
- pin boundary conditions (that is, restrained from translation in all directions) for both bearings at one end and roller boundary conditions (similar to "pin" except permitted to move in the bridge longitudinal direction) for both bearings at the other end.
- pin connections used for internal member-to-member connections.
- a 3-D skeleton analysis (although some engineers still use 2-D analysis of the vertical trusses and of the top and bottom horizontal trusses and combine the results). The skeleton model includes structural members but ignores other features, such as the deck or the railings (Fig. 6).

After the initial standard analyses were completed, the skeleton models were modified to include the stiffening effect of their respective decks and then analyzed again. Frame elements that represented stringers and plate elements that represented the deck were added to the skeleton models (Fig. 7). Pinned joints also approximated the stringer-to-floor-beam connection. The frame elements that represent the stringers were offset from floor beams to represent the stacking of actual stringers on the floor beams (Fig. 8). The plate elements were offset in a similar manner, again to represent the stacking of actual deck elements on the stringers (Fig. 9).[11]

It is important to note that all five of the bridges had deck elements layered one on top of the other: the deck is above the stringers, and the stringers are above the floor beams. It would be incorrect to model all of these elements in a single plane, as that would overstate the stiffness of the deck system. Because frame elements and plate elements lie in different horizontal planes, a modeling contrivance in the form of offset elements — specifically, frame members inserted between the centerlines of the floor beams and stringers and again between the centerlines of the stringers and the deck elements — was used to connect these members (Fig. 10). There are no real offset members in the bridges; these members in the model serve to connect the different frame elements that represent the floor beams and the stringers to one another, as well as to connect the frame elements that represent the stringers to the plate elements that represent the decks. As such, the offset members are modeled as weightless and stiffer than the elements that accurately represent bridge members. These artificial members have fixed ends at their connection to the "real" frame and plate elements; thus the deformations at the ends of the connected elements will be the same for strain compatibility. The offset members also have rotational releases where the "real" stringers interface with the floor beams and where the "real" deck interfaces with the stringers, because member rotation can occur at these locations.

Various connections were found in the five bridges, including connections that were:

- puddle welded, such as Prowers and Rifle bridges' corrugated deck to steel stringers.
- bolted, such as Blue River Bridge's deck planks to stringers.
- riveted, such as Prowers Bridge's stringers to floor beams.
- spiked, such as Fruita Bridge's deck planks to timber stringers.
- friction, such as Fruita Bridge's timber stringers to steel floor beams and San Miguel Bridge's gravel roadbed against corrugated metal pipe segments.

For all of these cases, pin joints were modeled to improve the accuracy of the analysis because it was believed that member rotation could occur at these connections. Although the physical constructions may be complex, the models for this second analysis are relatively simple.

Finally, a third analysis was completed after the models were modified to treat the decks as structural diaphragms, that is with in-plane rigidity, by mathematically locking the plate joints in the plane of the deck from deformation. This change further reduces the axial forces in the bottom-chord eye-bars. The diaphragm model is presented as a potential upper bound for lateral stiffness of the deck.

These techniques are described in greater detail in the report on a research

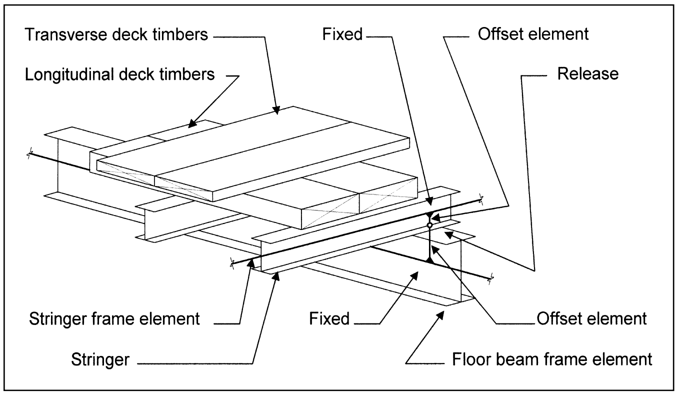

Fig. 8. Typical offset members and release locations. The rotational release point is located at the intersection of the bottom of the stringer and the top of the floor beam. This is a representation of Blue River Bridge.

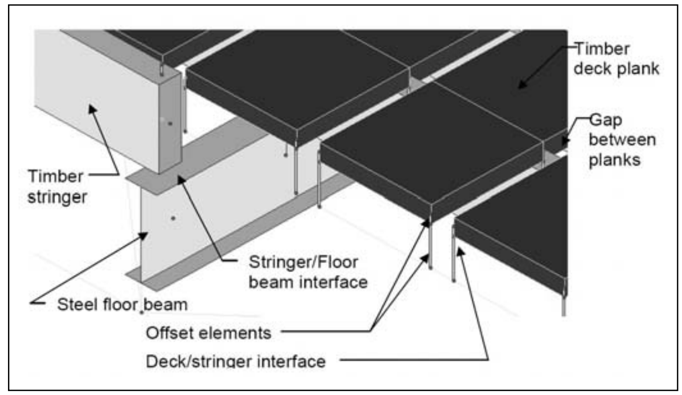

Fig. 9. Computer-generated rendering of the timber deck planks on timber stringers on steel floor beams, including offset elements. There is a rotational release at the location of the deck/stringer interface. This model is for the Fruita Bridge deck, so gaps are present between the rows of plate elements.

project completed by the Department of Civil Engineering at the University of Colorado at Denver, as well as in other sources cited here.[12]

Timber Decks

Two of the bridges had timber decks, Fruita Bridge and Blue River Bridge. However, configurations of the decks were different. Each 155-foot (47-meter) span of Fruita Bridge has eight bays, with steel floor beams and timber stringers covered by a timber deck (Fig. 1). Steel eye-bars serve as bottom chords and principal diagonals, and steel rods provide counterbracing and cross-bracing in the plane of the top and bottom chords. The bridge served highway traffic from 1907, when it was constructed, until a replacement bridge was built about one-half mile downstream in 1970. The bridge has been abandoned since then. The City of Fruita would like to reopen the bridge for pedestrian and bicycle use as part of a bikeway leading to nearby tourist attractions but has been stymied by the expense of rehabilitation.

The pin-connected, skeleton-frame model of Fruita Bridge was analyzed under AASHTO loads. For the deck model, individual deck planks, with gaps between the planks, were approximated (Fig. 9). The actual deck planks are spiked to the timber stringers, so the model approximated the spiked connection as pinned. The modulus of elasticity for the wood deck planks was input directly, as was thickness of the planks. The moment of inertia was not input directly because the stiffening effect of the plank's geometry is accounted for in the finite-element analysis by the software. The diaphragm model was then analyzed as a potential upper bound for stiffness, although it is considered unrealistically stiff in the lateral direction because of its in-plane rigidity, which actual wood decks with gaps between the planks clearly do not possess.

The other timber-deck bridge, located over the Blue River near Silverthorne and Dillon, Colorado, is believed to have been built in about 1895 as Two-Mile Bridge, near Breckenridge, Colorado, and moved to the Blue River site at an unknown date (Fig. 2). This steel Pratt truss has five bays and a span of 80 feet (24 meters), with a timber deck consisting of longitudinal "running boards" on transverse planks on steel stringers. The steel stringers bear on and are mechanically attached to the steel floor beams. The bridge has steel eye-bar bottom chords and diagonals and steel-rod cross-bracing at the center bay. The railing is a steel lattice with double-angle top and bottom rails.

While it has transverse deck planks similar to Fruita Bridge, longitudinal running boards have been added on top of the deck planks. The orthogonal crisscrossing of running boards and deck planks creates a much more continuous deck than that at Fruita Bridge. The two layers of mutually orthothropic timbers, well spiked together, approximate a single solid deck. The wood's modulus of elasticity was input directly in the deck model, as was plank thickness. Again, there was no need to input the moment of inertia, as the software's finite-element analysis accounts for stiffness due to geometry. The behavior of interconnected deck elements is quite different from the deck at Fruita Bridge, which had gaps between the individual deck planks. The deck of Blue River Bridge was modeled with a grid of interconnected plate elements, which were connected to the supporting stringers with rigid offset frame elements (Fig. 10). One might expect this virtually solid deck to behave more closely to a rigid diaphragm than the deck at Fruita Bridge. This theory was confirmed by the analyses.

Corrugated-Metal Decks

Three of the bridges had had their original timber decks replaced with corrugated metal decks. Prowers and Rifle bridges have corrugated-steel bridge decks topped with asphalt pavement. San Miguel Bridge has an unusual configuration of semicircular segments of corrugated-metal pipe topped with gravel roadbed. These three decks were all much heavier than the original timber decks.

Prowers Bridge over the Arkansas River, near Lamar, Colorado, consists of six spans of various constructions (Fig. 3). The span selected for study was a 160-foot (49-meter) camelback Pratt through truss that was built in 1909 and abandoned in 1994, chosen because it

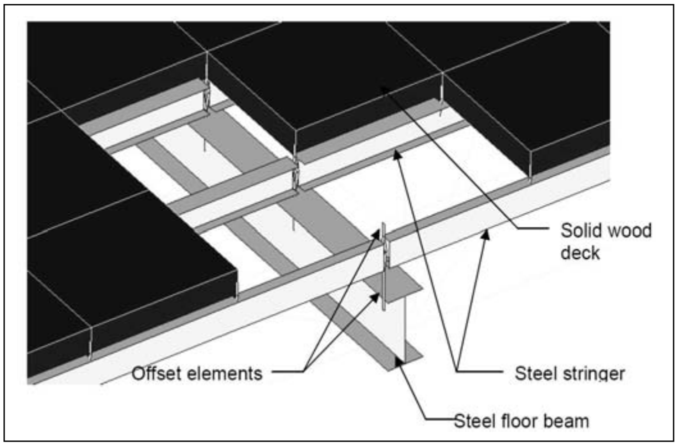

Fig. 10. Computer-generated rendering of timber deck on steel stringers on steel floor beams. The deck has been modeled using plate elements interconnected at all their nodes (corners) and frame elements for the stringers and floor beams. The offsets are weightless "dummy" frame elements of high stiffness, used for connectivity only. This model is of the Blue River Bridge deck.

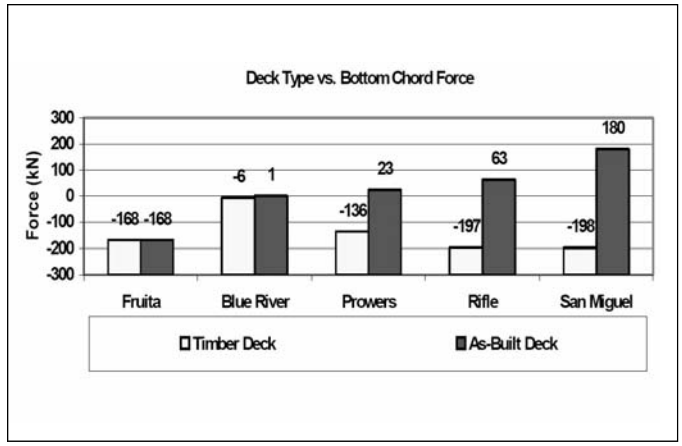

Fig. 11. Windward bottom-chord force for timber decks and as-built decks. For all cases, the windward bottom chord is in compression for the timber deck cases and would require significant cost to remedy. Fruita Bridge has the same values for both cases because the existing (as-built) deck is the same configuration as its original timber deck. The other bridges have higher forces in the windward bottom chord, because the higher as-built deck weights increase tension in the bottom chords.

was the longest span. It has steel eye-bar bottom chords and diagonals with steel-rod counterbracing. The railing is a steel lattice with single-angle top and bottom rails. Rifle Bridge, over the Colorado River at Rifle, consists of two spans (Fig. 4). The 240-foot (73-meter) Pennsylvania truss was selected for study because it was the longer span. It has steel floor beams with steel stringers, steel eye-bar bottom chords and diagonals, and steel-rod counterbracing. The railing is a steel lattice with double-angle top and bottom rails. It has been abandoned since the late 1960s, when a replacement bridge was constructed.

For both Prowers and Rifle bridges, the corrugated-metal-and-pavement deck construction was modeled with interconnected elastic plate elements similar to Blue River Bridge. This is a simplification: the corrugated metal is much stiffer in the direction of the flutes and more flexible transverse to the direction of flutes. Further complicating the modeling is the fact that high flexural stresses in the bridge's lateral direction, induced by wind pressure, would tend to occur near mid-span, where the deck was most flexible laterally, but high shear stresses would tend to occur near the span ends where the deck was considerably stiffer laterally. The lateral flexibility of the corrugations leads to a greater sensitivity to stress in the longitudinal direction, but not nearly so in the transverse direction.[13] Because of software limitations, the deck was modeled with a grid of elastic plate elements of constant stiffness in all directions, as if the deck were an isotropic solid (Fig. 10).[14] Values for the modulus of elasticity were input for plate and shell elements and apply in all directions; thus, this simple approach to a complex problem was adopted. Despite this limitation, it was felt that a methodology utilizing readily available software tools would be more beneficial to preservation efforts than the use of more expensive software with greater analytical precision. However, different stiffnesses were studied in the course of analysis, and the stiffness with the best fit to field-acquired test data was adopted.[15] Prowers Bridge was modeled using RISA 3D, and Rifle Bridge was modeled using RAM Advanse.

San Miguel Bridge had been constructed originally with a timber deck on timber stringers with five spans as Fifth Street Bridge over the Colorado River at Grand Junction in 1886. When that bridge was replaced in the 1930s, one of the spans was relocated to the San Miguel River site (Fig. 5). This wrought-iron bridge with a 142-foot (43-meter) span was subject to heavy live loads from ore-carrying trucks in an active mining region of the Colorado Plateau. The current deck, consisting of gravel roadbed on semicircular segments of corrugated-metal pipe supported on steel stringers, was installed in 1964. Gravel roadbase, identical to that used on road, was placed over the pipe segments. The thickness of the gravel roadbase varied from about 4 inches (above the apex of the pipe segments) to about 16 inches (above the interface of the corrugated metal pipe segment and the bottom flange of the stringers). This construction results in a very heavy deck. At approximately 74 psf (3.54 kPa), the San Miguel deck had the highest of all the deck dead loads studied. This deck was modeled using RISA 3D, and interconnected plate elements were used to represent the gravel roadbed. One change from the previous example is that the offset elements were modeled so that the deck elements were in the same plane as the stringer top flanges. This change in the model is relatively minor, although the physical deck construction was quite different. As with the Prowers and Rifle models, this decision was made for modeling simplicity.

Comparisons

Historically, these bridges were built with timber decks. Fruita Bridge still has a timber deck, albeit a replacement, in its original configuration. Longitudinal running boards were added on top of the transverse deck timbers of Blue River

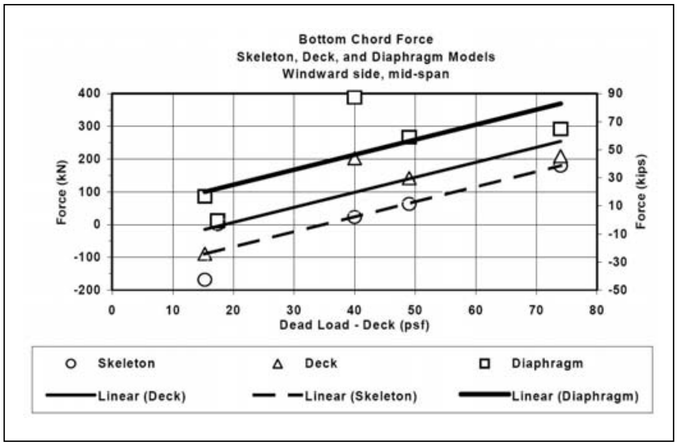

Fig. 12. Force versus deck dead load for skeleton, deck, and diaphragm models. A linear regression curve for each data set from five different bridges is shown. For a given deck dead load, the skeleton models show the least bottom-chord force, which represents compression, if negative. The deck models show higher bottom-chord forces, considered to be more realistic. Finally, the diaphragm models show a theoretical, albeit unrealistic, upper bound for bottom-chord force.

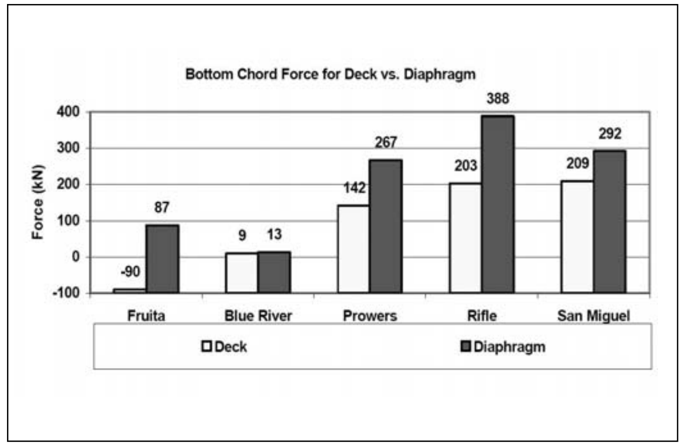

Fig. 13. Windward bottom-chord force versus deck dead load for deck and diaphragm models. Where there are only small differences between the deck and diaphragm values, the deck is about as laterally stiff as theoretically possible, e.g., Blue River Bridge. Large differences indicate that the deck is not nearly as stiff as theoretically possible, such as at Fruita Bridge.

Bridge. The other bridges have replacement decks of other configurations, all heavier than the original timber deck.

In the models discussed here, the alternative load path of the deck as a lateral stiffening feature has been introduced. It is concluded that the combination of skeleton and deck reveals the stiffness of the bridge in the lateral direction, resulting in a significant reduction of axial forces calculated in the bottom-chord eye-bars as compared to those calculated using a traditional skeleton model. The increased lateral stiffness due to the deck's contribution to the total structure reduces forces otherwise determined by skeleton analysis in many members. A measure of this effect can be found in the midspan bottom-chord members. These members have been selected as an example of this effect because: they respond to wind by developing relatively greater force than other members; analysis of skeleton structures often indicates undesirable compression in these members; and these members were selected for instrumentation in field tests and thus field measurements could be compared to analytical results. As this paper focuses on modeling, the field test results are not presented; they can be found in the references.[16]

Wind pressure against a truss bridge results in tension forces being developed in the leeward members and compression forces in the windward members. Net compression can occur in the windward bottom chords if the wind-induced compression exceeds the self-weight-induced tension. For eye-bar members intended for tension only, the net compression calculated by the skeleton method is often sufficient to result in the buckling of the member. (Note that the original design wind pressure was probably much lower than today's requirement, as discussed above. Bottom chords that are eye-bars — the most common type — were originally designed for net tension under the load combination of self-weight plus wind.) Thus, the bottom chords are of particular importance to a truss bridge. The upper chords, subject to compression under self-weight, will have increased compression on the windward side as well. The construction of upper chords was originally designed for compression; for the bridges discussed here the increased net compression, even for the skeleton analyses, fell within the capacity of the as-built upper chords.

Figure 11 shows the relationship of the force in the windward bottom chord for decks of different dead loads. Forces from analyses with the original timber decks, with relatively light dead loads, are plotted next to forces from analyses made using the current and higher deck dead load. The problem of compression in the windward bottom chords is clearly more pronounced in the lighter, timber-deck models.

Figure 12 summarizes the findings for the case of axial force in the midspan bottom chord on the windward side. The results from the skeleton, deck, and diaphragm models for all five bridges are examined. Two correlations can be seen: first, a higher deck dead load results in increased tensile force in the bottom chords, and secondly, the use of deck models instead of skeleton models also results in increased tensile force in the bottom chords. Increased tensile force in the bottom chords reduces, or in many cases eliminates, the problematical compression in the windward bottom chord. The diaphragm models, in which the plane of the deck was locked to prohibit deformation, were included, not because they are considered realistic, but because they represent theoretical upper bounds, least compression or highest tension, on bottom chord force (i.e., on deck lateral stiffness).

Figure 13 shows the relationship of the force in the windward bottom chord for the deck models versus those for the diaphragm models. The greatest difference between the deck and diaphragm models was at Fruita Bridge, where the deck was discontinuous. This condition demonstrates an advantage in using continuous decks, although that structural advantage must be balanced against the use of historically accurate timber deck planks.

Further information on the authors' work on modeling of this behavior can be found in the references.[17]

Conclusions

Use of skeleton models will lead to artificially low bottom-chord forces (i.e., artificially high compressive forces). The problem of high calculated compression in windward bottom-chord eye-bars under the combination of dead load plus wind load can be addressed in two ways:

- Account for the stiffening effect of the deck. As the deck stiffens the structure, the windward bottom-chord force decreases in compression or increases in tension. Use of deck models will more accurately predict the actual bottom-chord forces. Depending on the deck dead load, this change may be sufficient to remedy the problem of eye-bar compression. The series of linear regression curves shown in Figure 12 illustrate this effect. Note that there is no construction cost; this approach is entirely analytical.
- Add dead load to the deck. Increasing the deck dead load increases the tensile force in all bottom chords. There is an upper bound to the amount of additional dead load: at some point member stresses under live and dead loads will be limiting. Figure 11 illustrates this effect. There will be construction costs.

An analytical model that includes the deck with the skeleton can account for the stiffness of the bridge in the lateral direction, resulting in a significant reduction of axial forces calculated in the bottom-chord eye-bars compared to those calculated using a traditional skeleton-only model.

FREDERICK R. RUTZ, PhD, PE, is a senior project manager at J.R. Harris & Company in Denver and has been a practicing structural engineer since 1972. His 2004 doctoral dissertation research at the University of Colorado at Denver focused on the stiffening effect of timber decks for truss bridges.

KEVIN L. RENS, PhD, PE, is an associate professor in civil engineering at the University of Colorado at Denver, where he has taught since 1995. His specialties are inspection, rating, maintenance, and repair of infrastructure; nondestructive evaluation and failure analysis of infrastructure; and properties of cement and concrete.

VERONICA R. JACOBSON is a graduate student in structural engineering and a research assistant in the Department of Civil Engineering, University of Colorado at Denver.

SHOHREH HAMEDIAN is a structural engineer at JR Engineering, Greenwood Village, Colorado, and a graduate student in structural engineering at the University of Colorado at Denver.

KAZWAN M. ELIAS, EI, is a structural engineer and a graduate student in structural engineering at the University of Colorado at Denver.

WILLIAM B. SWIGERT, PE, SE, is a senior structural engineer at Schmueser Gordon Meyer, Glenwood Springs, Colorado, and a graduate student in structural engineering at the University of Colorado at Denver.

Acknowledgements

The University of Colorado at Denver has conducted this work funded in part by Grant No. MT-2210-04-NC-12 from the National Center for Preservation Technology Transfer and Grant No. 2004-M1-019 from the State Historical Fund of the Colorado Historical Society. Further, the cooperation of the City of Fruita, Summit County, Bent County, Montrose County, and Garfield County is gratefully acknowledged.

Notes

1. Eric DeLony and Terry H. Klein, "Rehabilitation of Historic Bridges," *ASCE Journal of Professional Issues in Engineering Education and Practice* 131, no. 3 (2005): 178.

2. Frederick Rutz, "Lateral Load Paths in Historic Truss Bridges" (PhD diss., University of Colorado at Denver, 2004), 127–130.

3. American Association of State Highway and Transportation Officials, *Guide Specifications for Design of Pedestrian Bridges* (Washington, D.C.: AASHTO, 1997).

4. Frederick Rutz inspected 16 pin-connected truss bridges in 2001. All were found all to suffer from varying degrees of deterioration and damage, but there was no evidence of wind-induced distress at any of the bridges. Fifteen bridges were in Colorado: Keystone Bridge, Bailey; Blue River Bridge, Summit County; Silverthorne Pedestrian Bridge, Silverthorne; South Canyon Bridge and Hardwick Bridge, Garfield County; Fruita Bridge, Fruita; Paonia Bridge, Paonia; San Miguel Bridge, Montrose County; Lado del Rio Bridge, Archuleta County; Costilla Crossing Bridge, Conejos and Costilla counties; Timpas Bridge, Timpas; Smith Hollow Bridge, Manzanola; Nyberg Bridge, Avondale; Sante Fe Avenue Bridge, Pueblo; and Larimer County Fairgrounds Bridge, Loveland. The sixteenth, Ft. Laramie Bridge, is at the Ft. Laramie National Historic Site in Wyoming.

5. Squire Whipple, *A Work on Bridge Building: Consisting of Two Essays, The One Elementary and General, The Other Giving Original Plans, and Practical Details for Iron and Wooden Bridges* (Utica, N.Y.: H. H. Curtis, 1847).

6. Joseph Pullaro, "Rehabilitation of Two 1890s Metal Truss Bridges," in *International Engineering History and Heritage*, 215 (Reston, Va.: ASCE, 2001). Pullaro offers a reason for these observations: "Trusses generally experience lower stresses than shown by analytical methods due to the overall composite action of trusses and the deck which is not accounted for in analytical methods."

7. David Fischetti, "Conservation Case Study for the Cornish-Windsor Covered Bridge," *APT Bulletin* 23, no. 1 (1991): 22–28. While the article discusses reconstruction work on the main lattice trusses, it does not describe the ingenious use of the deck in the lateral force-resisting system.

8. See Table 1 for wind pressures recommended by J. A. L. Waddell in *The Designing of Ordinary Iron Highway Bridges* (New York: John Wiley & Sons, 1884). For bridges in "unusually exposed" situations, Waddell recommended increasing these pressures by 10 psf (480 Pa). He advises applying this additional pressure to the vertical projected area of the floor and stringers and to twice the area of the vertical projection of the windward truss, railings, curbs, and ends of floor beams. It is assumed that his use of "twice the area" was meant to include the respective areas on the leeward side.

Fourteen years later, Waddell, writing in De Pontibus, had developed his "General Specifications Governing the Designing of Steel Highway Bridges and Viaducts." Three notable developments had occurred in the intervening years:

- With the commercial development of the Bessemer and open-hearth processes, steel had come of age. Steel was now the material of choice, having replaced wrought iron.
- Numerous highway-bridge failures had occurred. Wind had contributed to some of them.
- New bridges were virtually always of the Pratt or Warren types.

By 1916, when Waddell's classic *Bridge Engineering* was published, he had further refined his thoughts on wind loadings. He offered a more theoretical basis:

$$P = K\,V^2$$

where P is pressure in psf, K is a coefficient discussed below, and V is wind speed in mph.

Waddell explained that the coefficient K "cannot be given with any certainty, but is generally considered to lie between 0.003 and 0.005, with most of the later writers assuming it as 0.004 or less." Today we would treat K as equal to 0.00256 only for the stagnation pressure at sea level, not the design pressure. For objects with a drag coefficient of 2, which is an approximate average for common structural shapes today, the resulting K would be 0.00512, or slightly greater than Waddell's upper bound recommendation.

For railroad structures, Waddell would combine the wind pressures listed in Table 1 on both the structure and the train. For highway bridges, however, he would not combine them for the reason that "no person would ever venture upon the structure when there exists a wind pressure per square foot of anything like thirty (30) pounds."

9. Frank Hatfield, "Engineering for Rehabilitation of Historic Metal Truss Bridges," in *Proceedings of the 7th Historic Bridges Conference*, 7–11 (Cleveland: Cleveland State University, 2001).

10. RISA-3D, version 4.5, RISA Technologies, Foothill Ranch, Calif., 2001. RAM Advanse, version 7.0, RAM International, Carlsbad, Calif., 2005.

11. These elements are four-joint (quadrilateral) elements. They are called "mixed interpolation elements" because they are based on plate assumptions with added interpolating functions for out-of-plane shear. This approach is analogous to incorporating shear deformation with flexural effects from beam theory, resulting in an element that can be used for thin- and thick-plate applications. See "Plate/shell element formulation," RISA-3D, in the Help menu on CD-ROM. A reference for this element is K. J. Bathe, *Finite Element Procedures* (Englewood Cliffs, N.J.: Prentice-Hall, 1996), 420–449.

12. Frederick Rutz, Kevin Rens, Veronica Jacobson, Shohreh Hamedian, Kazwan Elias, and William Swigert, *Load Paths in Historic Truss Bridges*, No. 2004-25, prepared by Dept. of Civil Engineering, University of Colorado at Denver for National Center for Preservation Technology and Training, Natchitoches, La., under Grant No. MT-2210-04-NC-12, 2005, 14–56. Veronica Jacobson, "Analytical Techniques and Field Verification Method for Wind Loading Analysis of the Historic Prowers Bridge" (master's thesis, University of Colorado at Denver, 2006), 10–25. Rutz, "Lateral Load Paths in Historic Truss Bridges," 165–180. Shohreh Hamedian, "Analysis and Testing of the Historic Blue River Bridge Subjected to Wind" (master's thesis, University of Colorado at Denver, 2006), 46–60. Frederick Rutz, Kevin Rens, Veronica Jacobson, Shohreh Hamedian, Kazwan Elias, and William Swigert, "Response of Pin-Connected Truss Bridges to Wind," *Proceedings of the 2006 Structures Congress, Structural Engineering Institute of the American Society of Civil Engineers, May 17-20, St. Louis, Mo.*, CD-ROM (ASCE, 2006). Teby Hererro, Frederick Rutz, and Kevin Rens, "Field Testing and Data Acquisition of Historical Truss Bridges using Modular Strain Transducers," *Proceedings of the 2006 Structures Congress*, CD-ROM. Frederick Rutz and Kevin Rens, "Alternate Load Paths in Historic Truss Bridges: New Approaches for Preservation," *Proceedings of the 2004 Structures Congress, Structural Engineering Institute of the American Society of Civil Engineers, May 22-26, Nashville, Tenn.*, CD-ROM (ASCE, 2004).

13. Jacobson, 18–23.

14. The versions of RISA 3D and RAM Advanse apply the modulus of elasticity in all directions. These software packages were used intentionally because of their widespread availability and use. More sophisticated (and expensive) software (SAP, for example) has the ability to accept different values for the modulus of elasticity in different directions.

15. Jacobson, 74–77.

16. Rutz et al., *Load Paths in Historic Truss Bridges*, 60–63, 68–71, 76–79, 83–86, 91–93. Jacobson, 53–67. Rutz, "Lateral Load Paths in Historic Truss Bridges," 181–225. Rutz et al., "Response of Pin-Connected Truss Bridges to Wind," 6–9. Hamedian, 24–40, 61–72.

17. Rutz et al., *Load Paths in Historic Truss Bridges*, 57–59, 64–68, 72–76, 80–82, 87–89, 94. Jacobson, 68–78. Rutz, "Lateral Load Paths in Historic Truss Bridges," 226–284. Hamedian, 61–72.