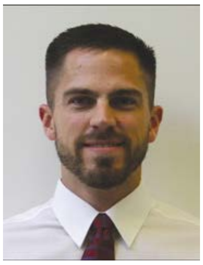
Live Load Testing of Historic Covered Timber Bridges

Travis Hosteng
Research Engineer
Bridge Engineering Center
Iowa State University
Ames, IA United States
kickhos@iastate.edu



Travis earned his MSc degree from Iowa State University. He is the lead engineer for wood bridge projects at the BEC. He is also a member on the ASCE committee for timber bridges.

James Wacker
Research Engineer
Forest Products Laboratory
Madison, WI United States
jwacker@fs.fed.us

Jim earned his Civil Engineering degree from the University of Wisconsin. He is a registered professional engineer with more than 20 years' experience in timber bridge research.

Brent Phares
Director
Bridge Engineering Center
Iowa State University
Ames, IA United States
bphares@iastate.edu

Brent is the Director of Iowa State University's Bridge Engineering Center. He is regarded as an international expert in the field of advanced bridge evaluation technologies.

Summary

The National Historic Covered Bridge Preservation Program (NHCBP), sponsored by the Federal Highway Administration (FHWA), is intended to preserve covered timber bridge structures nationwide. Today, less than 700 covered timber bridges still exist in the United States and of those many are closed to vehicular traffic. Furthermore, a large percentage of the remaining bridges open to traffic are restricted by load postings. Currently, no load rating standards exist for covered timber bridges which often results in overly conservative load ratings/postings. Rating engineers simply do not have adequate resources or background in timber design, especially in covered timber bridge design, for calculation of accurate load postings on covered timber bridges. To better understand the structural performance of covered timber bridges and ultimately develop improved criteria for load rating of these bridges, a series of field tests were performed on eleven historic covered timber bridges. Field live load testing was conducted on three Burr arch truss bridges in Indiana, four Howe truss bridges in Indiana, and four Queen post truss bridges in Vermont. This paper discusses the live load testing of the Cox Ford Burr arch Covered Bridge in Indiana. Included in the discussion is the procedure for installing displacement and strain sensors on the bridge, monitoring those sensors for global displacements and member strains at various cross-sections during passage of a known test vehicle, the results from the load test, and information on how these field tests were used to develop and calibrate an analytical model. The analytical model could then be used for determining more accurate bridge load ratings of the bridge. We believe this work will result in a viable technique for determining reliable load ratings for covered timber bridges using field tests and more accurate computational modeling techniques.

Keywords: Covered Bridges, Timber, Load Rating, Burr arch, Queenpost.

1. Introduction

The Federal Highway Administration (FHWA), in partnership with the USDA Forest Products Laboratory and the National Park Service (NPS), has sponsored a comprehensive national research program on Historic Covered Timber Bridges in the USA. The main purpose is to develop improved methods to preserve, rehabilitate, and restore the timber bridge trusses that were developed during the early 1800's, and in many cases are still in service today. The overall goal of the National Historic Covered Bridge Preservation Program (NHCBPP) is to preserve these iconic bridge structures for future generations. The study described herein is part of the NHCBPP and is aimed at establishing a procedure for safely and reliably load-rating historic covered bridges through physical testing. To accomplish this goal, live load testing was conducted at three Burr arch truss bridges in the state of Indiana, four Queenpost truss bridges in the state of Vermont, and four Howe truss bridge in Indiana. The field testing results have formed the basis for improved instrumentation, load testing, and load rating of similar historic covered bridges in the future. This paper provides an overview of the field testing methods, results, model calibration and a brief overview of how the calibrated analytical model may be used for load rating procedures for the Cox Ford Burr arch Bridge in Indiana.

2. Background

Often, given the age and complex behaviour of timber bridges and especially covered timber bridges, they are assigned relatively low ratings. It is also widely known that, when tested, most bridges are found to possess structural characteristics that supersede those assumed for the assigned ratings which were determined using prudent engineering judgment. In general, these behaviours result from additional, unaccounted-for stiffness and improved load distribution characteristics. Although testing procedures have been established for conventional bridges, no such procedures have been established for historic covered bridges. Given the historic nature and unusual geometric features of these structures, a procedure needed to be established detailing how to safely and reliably determine load ratings for historic covered timber bridges through physical testing. Furthermore, the developed testing and rating procedure needed to be simple and general in nature such that practicing engineers have the ability to quickly and accurately analyze and assign safe load limits to covered bridges in their inventory with basic, off the shelf analysis software.

3. Methodology

In order to develop testing and rating procedures for historic covered timber bridges that are both accurate and easily applicable by practicing engineers, physical load tests were conducted on three groups of covered bridges with typical truss configurations, namely Burr arch, Queenpost and Howe trusses. These three truss configurations were selected because they have a significant population of structures surviving today. The physical load tests provided invaluable structural performance data, in addition to modeling and calibration parameter information for the basic 2D analytical models of the trusses for calculating load rating.

4. Cox Ford Burr arch Covered Bridge - Indiana

During October of 2010, three Burr arch covered timber bridges, Zacke Cox, Cox Ford and Portland Mills Bridges, were evaluated and tested in the State of Indiana. These bridges were all located in Parke County, IN, which maintains over 30 historic covered bridges within their roadway network. These single lane bridge structures are all currently restricted to vehicle weights below current legal loads, but still provide vital transportation links to rural communities in the western part of the state. Approximately 93 covered bridges exist within Indiana. The following outlines the testing and modeling of the Cox Ford Bridge. The Cox Ford Bridge (County Bridge No. 227) is located on Cox Ford Road (unpaved) and carries traffic over the Sugar Creek just west of Turkey Run State Park. An end view photograph of the bridge is shown in Figure 1.

Fig. 1 Interior view of the Cox Ford Bridge showing the Burr arch truss configuration

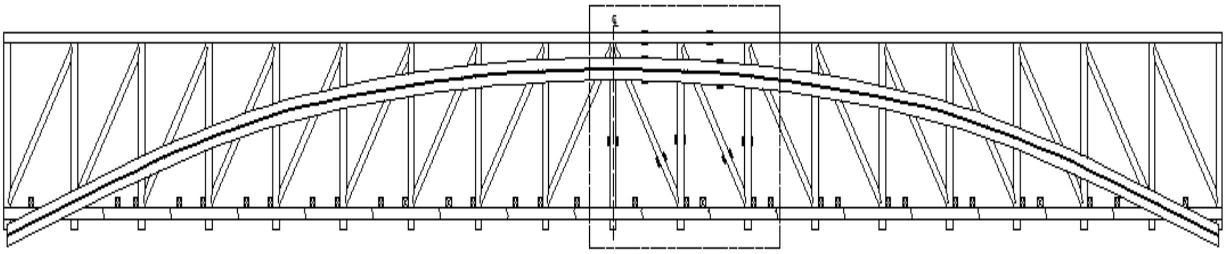
The Cox Ford Bridge was originally built in 1913 to replace an iron bridge and has been rehabilitated twice previously in 1975 and 1991. The structure is a one lane, single span, simply supported double Burr arch truss with a total length of 58.5 m (192 ft.). The bridge measures 52.9 m (173.5 ft.) from center-to-center of abutment bearing and the width of the bridge is 4.8 m (15.9 ft.) measured from out-to-out of the bridge deck. The total height from the bottom of the bottom chord to the top of the top chord was measured to be 5.4 m (17.8 ft.) and the average truss panel spacing is 3 m (9.75 ft.). The trusses consist of rectangular parallel chords sandwiched by double concentric arches, two member bottom chords with nine single-headed hook fishplate and iron shoe splice joints, single member upper chords, single member diagonals and single member verticals. The trusses and arches are interconnected with iron spikes/bolts at the vertical and diagonal members. Currently the structure is rated and posted for a 5 mton (5 ton) load limit. The following is a case study focusing on the field testing and analytical modeling results from the Cox Ford Bridge.

5. FIELD TESTING

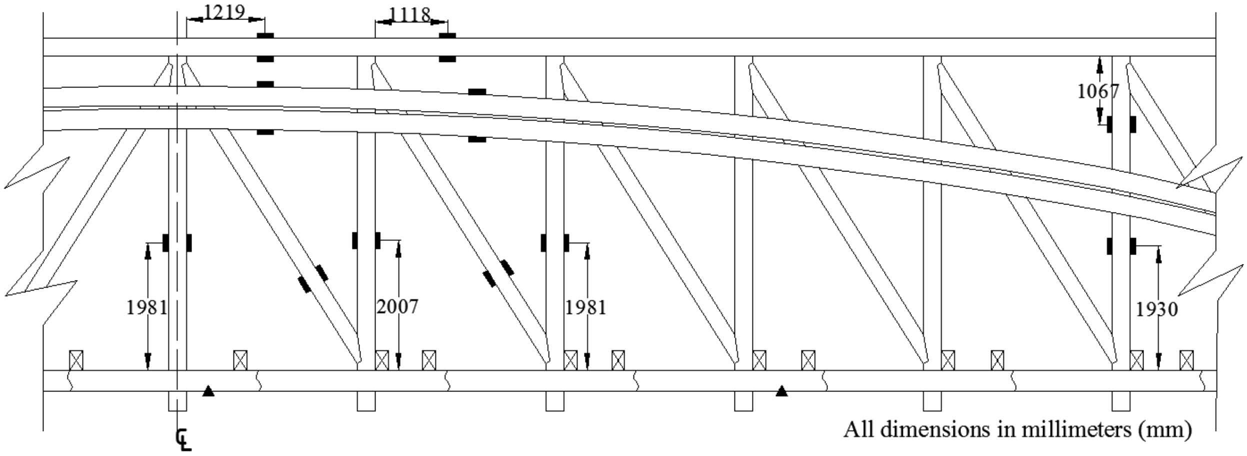
Testing of the Cox Ford Bridge involved installing displacement and strain transducers on the bridge at various cross sections and loading the structure with a vehicle of known weight.

Global displacements of the structure, specifically the trusses, were measured at the following locations on each truss (see Fig. 2 for the instrumentation plan used for the testing of the Cox Ford Bridge): midspan, and at the location of both bottom chord splices. These displacements were recorded with ratiometric displacement transducers mounted on tripods connected to the bridge via aircraft grade steel cable extensions and recorded with an Optim Megadac data acquisition system (DAS) along with a Dell laptop computer running TCS software.

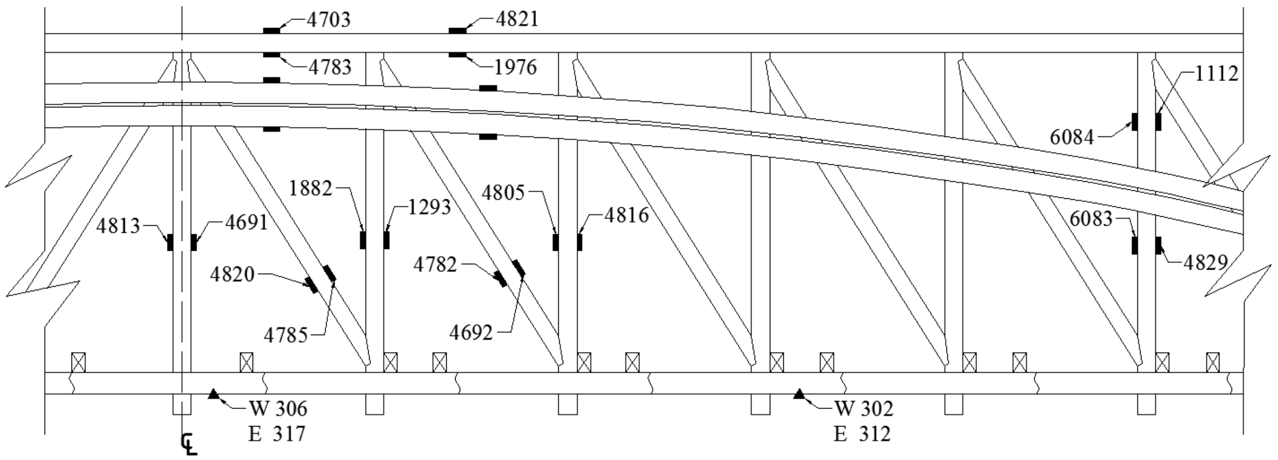
In addition to global displacements, member strains were recorded at various locations on the west truss using Bridge Diagnostics Inc. (BDI) DAS and BDI strain transducers. The strain transducers were attached to the timber members with hex-head screws and washers; in cases where aesthetics were an issue, smaller diameter drywall screws were used to minimize the holes left upon removal of the strain gage. Due to the limited number of gages available and time constraints, symmetry was assumed on the trusses and only one truss was instrumented for strain measurement (see Fig. 2 for locations of strain transducers).



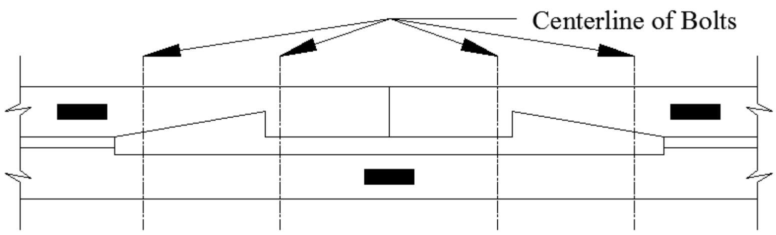
a. West truss instrumentation plan



b. Strain sensor locations



c. Strain sensor details



d. Typical chord joint strain sensor locations

Fig. 2 Strain sensor locations for the Cox Ford Bridge load testing

Loading for the Cox Ford Bridge was applied by driving two different trucks across the bridge: a full size dually pickup truck with a gross weight of 4,735 kg (10,440 lb.) (see Fig. 3) and a single axle dump truck with a gross weight of 8,528 kg (18,800 lbs.) (see Fig. 4).

Fig. 3 Small truck used for load testing the Cox Ford Bridge

Fig. 4 Large truck used for load testing the Cox Ford Bridge

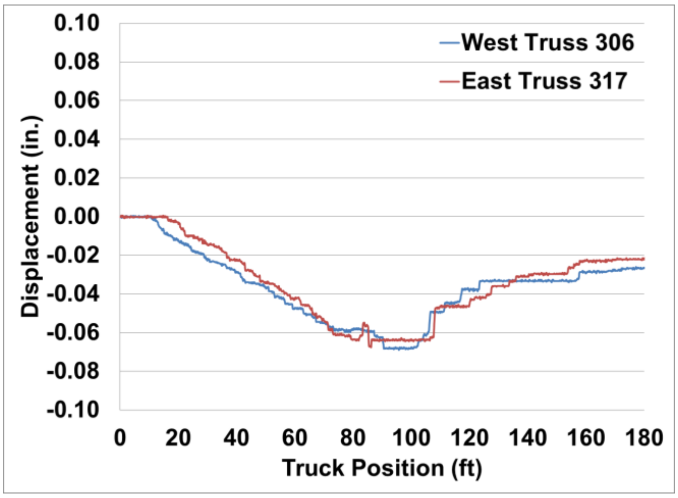
Three load paths were used during the testing of the Cox Ford Bridge. Load Case (LC) 1 involved the small truck centered on the longitudinal centerline of the bridge; LC 2 had the center of the small truck offset 610 mm (2 ft.) west of the longitudinal centerline of the bridge; and LC 3 was the large truck centered on the longitudinal centerline of the bridge. Given the narrow width of the bridge between the interior guardrails, no other load cases were deemed necessary.

6. Field Test Results

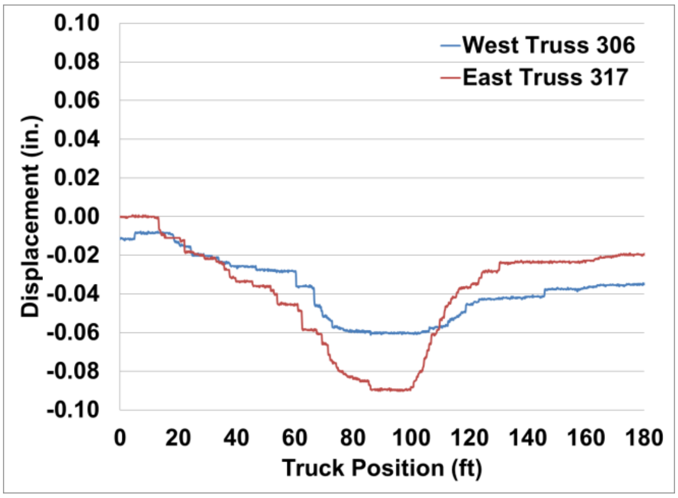
As noted previously, the main purpose in field testing the Cox Ford Bridge was to obtain measureable structural performance data that could be used to modify and calibrate an analytical model of the bridge; that analytical model would subsequently be used to perform the load ratings. The following section briefly discusses some of the test results and the key components evaluated during the testing process. Refer back to Fig. 2 for the locations of displacement and strain transducers discussed below.

6.1 Global Displacements

Figure 5 illustrates the displacements measured at the midspan of both truss bottom chords. A notable characteristic evident in Fig. 5 is that the bottom chord displacements are similar in behavior and magnitude. This indicates approximately equal load distribution transversely across the bridge deck to the trusses. Secondly, as the load is shifted away from centerline, LC 2, the magnitude of the deflection of the truss nearest the load becomes larger than the opposite truss, as expected. These transverse load distribution characteristics provide valuable information for the analytical model calibration process, and indicate that the structural response of the bridge is as expected. Furthermore, this means the field data are useful for the model calibration and load rating going forward.



a. LC1



b. LC2

Fig. 5 Global Displacements at Mid Span of the Bridge

6.2 Strains

Illustrated in Figs. 6 and 7 are the strain verses truck position plots for several cross sections of the Cox Ford Bridge. There are some key points of interest to be noted and carried over to the analytical model. First, since both strain gages on either side of the diagonal member essentially mirror one another in both direction and magnitude, the diagonal members may be classified as axial force members. Looking ahead, this gives clues as to the boundary conditions that may be needed in the modeling of the diagonals and verticals. Secondly, referring to the bottom chord splice strains, localized strain peaks are evident with the passage of each truck axle. This suggests that the bottom chord behaves more like a beam than an axial member when under loading, even though the truck/deck load is only transferred to the bottom chord as point loads at the locations of the transverse spreader beams. Again, this is critical information needed for accurate model generation. One final point to make from the two figures is that the structure is behaving elastically, i.e. the strains return to approximately zero after passage of the load truck. As a side note, instrumentation near the abutments would have been beneficial; had that been an option, then this data would have provided additional guidance in setting the initial level of end restrain in the analytical model.

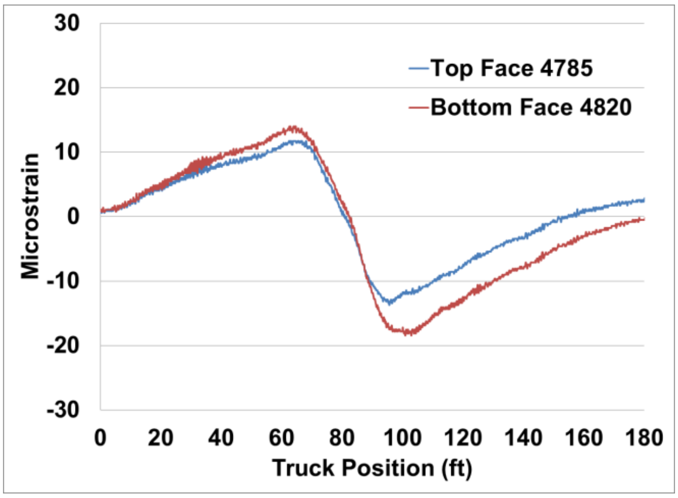


Fig. 6 West Truss Diagonal Between 9W and 10 W, LC1

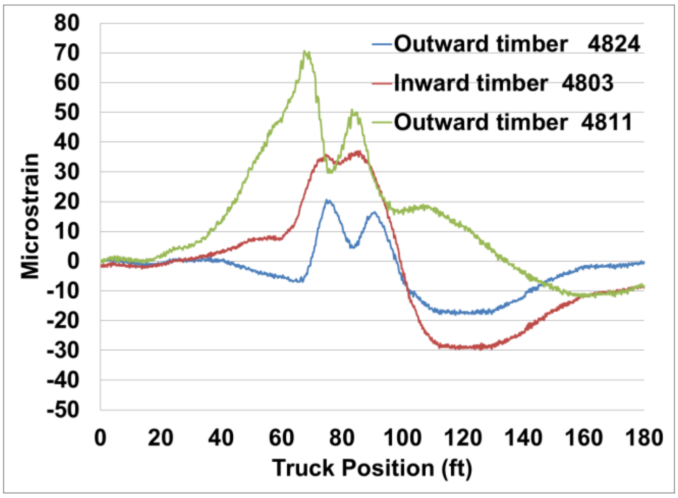


Fig. 7 East Truss Bottom of Bottom Chord, LC3

7. ANALYTICAL MODELING

7.1 Model Generation

Finite element models of historic covered timber bridges can be generated following the common linear-elastic modeling approach. Details on the modeling approach used for the Cox Ford Bridge are provided in the following sections. Information given below includes geometry definition, member properties, boundary conditions, end restraint decisions, and placement of live load. The finite element modeling software selected for this work was the program STAAD due to its familiarity and widespread use in the engineering community. Initial modeling of the structure began by creating a basic truss model of the structure, using truss elements for all members, pinned supports, and pinned connections between all members. The preliminary 2-D analytical model (Model 1) of the Cox Ford Bridge generated using STAAD is illustrated in Fig. 8.

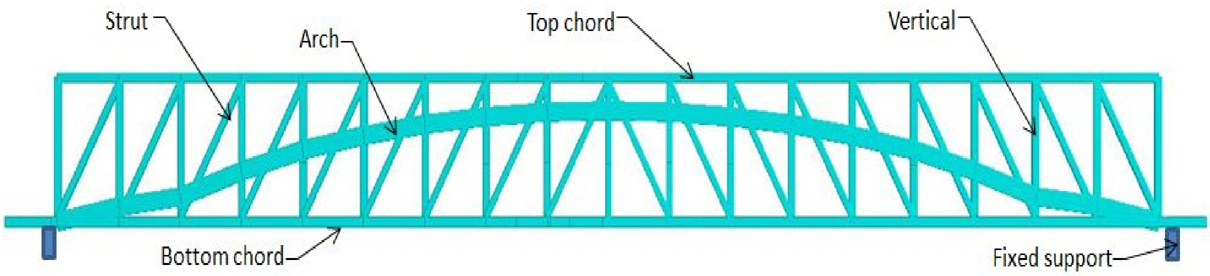


Fig. 8 STAAD model of the Cox Ford Bridge

Since the bridge was modeled in 2-D, and only one truss was modeled, transverse distribution was assumed to be 50 percent such that one wheel line went to each truss; this amount of transverse load distribution was verified (and discussed) earlier by the live load test results. The preliminary model was developed using two-dimensional beam elements for all primary structural members. Since as-built plans were not available, the model geometry was based upon field measurements taken during load testing. For convenience, the supports were defined to be at the end points of the bridge and initially assumed as pinned-pinned. Any built-in eccentricity between the diagonal and vertical members was assumed to be 152.4 mm (6 in.) (based on field inspection of joint details) to simplify the modeling process. The bottom chord was modeled as a monolithic, continuous member (i.e., neglecting the presence and location of member splice), and the connection between the top and bottom chord at the bearings was defined as rigid. All the primary load resisting components (bottom chord, top chord, diagonal and vertical) were defined as beam members whereas the arch members were defined to be compression members based on the knowledge of arch behaviour under loading and confirmed from the live load testing data. The arch members were connected to the truss by means of bolted connections to the verticals over its length and were initially assumed to be rigid. Because the model is two dimensional, geometric dimensions of the arch member were taken as the combined width and depth of the inner and outer arch rings. Furthermore, all members were assumed to be Southern Yellow Pine and the initial modulus of elasticity was assumed to be 0.333 kN/m^2 (1850 ksi) for all members of the truss in Model 1 [1]. After a model of the load truck used during testing was generated in the analysis software, calibration of the model was carried out by applying the truck loading scenarios to the model (in 30.5 mm (0.1 ft.) increments across the structure) along the same paths travelled during testing and comparing the analytical results with the results from the field testing.

7.2 Model Calibration

Initial results from running the analysis of the model and comparing the field and analytical strains on the pinned-pinned model were found to be less accurate than desired; therefore the bridge model end restraint conditions were modified to fixed-fixed in an attempt to bound the results. Strain comparisons from the fixed-fixed model indicated that the actual performance of the structure was somewhere between the pinned-pinned and fixed-fixed conditions, as expected. Subsequently, rather than incorporate complex joint fixity parameters and/or variable support restraints (springs) to improve the accuracy of the results, the decision was made to develop simpler methods of modifying the structure to obtain the best results while, for the most part, retaining the integrity of the structure. Again, the ultimate goal was to make the modeling process as straight-forward as possible.

Through trial and error (modifying member element types, member properties (E), member connection details, etc.) a 2-D model of the truss was developed that improved the accuracy of the results. The resulting STAAD model consists of fixed supports; pinned member connections; truss elements for the verticals, diagonals and upper chord; and beam elements for the bottom chord. These changes improved the correlation between the field test data and the analytical model results from approximately 40-50 percent to approximately 75-85 percent. Given the complexities inherent with timber as a material, along with the additional uncertainties input from the truss connection details and the presence of the framing/roof of the covered bridge, a percent error between the field and model results of approximately 20 percent is acceptable.

A side note: although the wall sheathing and roof of these covered bridges are not intended to be structural elements, they contribute to the stiffness of the structure in some regard. It should be noted that these and other unknown factors contributing to the performance of the actual structure were not represented in this STAAD model. These unaccounted for features likely contribute to the discrepancy between the field results and the analytical modeling results presented herein. It is felt, however, that ignoring these other sources of stiffness is a conservative assumption. The handling of these bridge characteristics in future modeling and rating of covered timber bridges is left to the discretion of the engineer of record.

8. LOAD RATING

The final step in the process is performing a load rating for the subject bridges using the calibrated FE model developed for that structure from the live load testing results discussed previously. The basic procedure for performing the load rating in this manner after generating a FE model are as follows: 1) based on the findings and recommendations from the previous section, create a FE model of the structure; 2) calculate and apply dead loads for the structure; 3) apply live loads, rating vehicles and/or different legal trucks, across the 2-D model to obtain member forces (noting that things such as vehicle height and width in addition to weight may control if the vehicle can safely enter and cross the covered bridge); 4) determine appropriate impact factor and input into model; 5) calculate member capacities (for all critical members); 5) analyze the model and calculate load ratings (ratio of member capacities to member forces output from FE model). The following briefly defines/outlines each step.

The initial step is calculation and application of the dead loads of the structure. Included in the dead loads are the weights of all bridge components, consisting of the primary (e.g., chord members, inclined posts, and vertical posts) and secondary components (e.g., side planks, roof, etc.). The dead loads are generally assumed to be uniform loads and can generally be calculated from the sectional properties for each component as measured in the field or from as-built plans and an assumed density of timber. Once the dead loads are calculated, they are applied to the calibrated model and the user moves on to live loads.

According to the AASHTO Load and Resistance Factor Rating (LRFR) manuals (AASHTO 2011), the AASHTO HL-93 truck and the AASHTO design tandem load combined with the standard uniform lane load should be used as the live loads to determine load ratings. The HL-93 truck and design tandem load are to be considered individually (with the standard uniform lane load). In all cases the HL-93 truck and tandem load should be placed 610 mm (2 ft.) from each curb as well as located at all other critical transverse locations (AASHTO 2004; 2010).

Along with the live load, consideration of the impact factor to be used for the particular bridge must be established. According to the AASHTO LRFD Specification (AASHTO 2004; 2010), a 33 percent increase in live loads for typical bridges can be utilized for dynamic load allowance (IM). However, the IM of timber components used for rating purposes is to be reduced by one half (to a maximum of 16.5 percent) according to the AASHTO LRFR Manuals (AASHTO 2011).

Once all the dead and live loads have been determined and entered into the model, calculation of member capacities is performed to complete the ratings. Typical historic covered bridges consist of members in tension and/or compression and members in combined flexure and axial loading; thus, the axial and/or flexural capacities should be determined separately in accordance with the appropriate AASHTO LRFD Specifications (AASHTO 2004; 2010).

After the member capacities are computed, the loading is performed to obtain the member strains that result from the rating vehicles. These member strains are then compared to the member capacities to obtain the load rating. Load ratings for members not exposed to combinations of loading types (including tension/compression members, flexure members, members under shear) may be computed as outlined in the AASHTO LRFR Manuals (AASHTO 2011).

During analysis of the live load field test data from the covered timber bridges, strain responses of certain members indicated they act like beam-column members because they resist combined flexure and axial loads; for Burr arch trusses those members are the top chord, bottom chord, and verticals. Therefore, load ratings reflecting the effects of the combination of flexure and axial loads should be considered for these members. Load ratings for the beam-column members can be estimated using design-based interaction equations provided by the AASHTO LRFD Specifications (AASHTO 2004; 2010).

It should be noted that the current AASHTO LRFD (2010) Specifications provide the interaction equation that has been modified to take into account the Euler buckling stress and gross cross-sectional area.

9. Future Work & Acknowledgements

Phase I of this work involved live load testing 11 covered timber bridges, divided into three groups of different truss configuration, namely Burr arch, Queen post and Howe trusses. The collected field test data along with a preliminary finite element analytical model of each bridge were used to calibrate each model to replicate the structural response of the bridges. These calibrated models could then be utilized to perform load ratings on each bridge for various vehicle configurations. Currently there are more than 3 truss types that make up the approximately 600-700 covered timber bridge still in service in the United States. Phase II will seek out an additional 2 or 3 common truss types of covered timber bridges for live load testing and analysis such that recommendations for modeling and load rating of these types of bridge through physical load testing may be added to those from Phase I.

This study is part of the Research, Technology and Education portion of the National Historic Covered Bridge Preservation (NHCBP) Program administered by the Federal Highway Administration. This study is conducted under a joint agreement between the Federal Highway Administration – Turner Fairbank Highway Research Center, and the Forest Service – Forest Products Laboratory.

10. References

[1] Ross R.J., White R.H., Pellerin R.F., Wang X, and Barshaw B. K., *Wood and Timber Condition Assessment Manual, 2010.*

[2] American Association of State Highway and Transportation Officials (AASHTO), *The Manual for Brigde Evaluation, AASHTO, Washington, D.C., 2011.*

[3] American Association of State Highway and Transportation Officials (AASHTO), *LRFD Bridge Design Specifications, AASHTO, Washington, D.C., 2010.*