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FIELD INVESTIGATIONS OF HISTORIC COVERED TIMBER BRIDGES IN THE USA

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ABSTRACT: The Federal Highway Administration is sponsoring a comprehensive research program on Historic Covered Timber Bridges in the USA. This national program's main purpose is to develop improved methods to preserve, rehabilitate, and restore the timber bridge trusses that were developed during the early 1800s, and in many cases are still in service today. The overall goal of the National Historic Covered Bridge Preservation Program is to preserve these iconic bridge structures for future generations. One of the many ongoing studies is aimed at establishing a procedure for safely and reliably load-rating historic covered bridges through physical testing. This paper will provide an overview of the field testing methods, results, and finite element modeling procedures and briefly discuss the load rating procedures to be developed based on the testing results.

KEYWORDS: Covered Bridges, Timber, Load Rating, Burr-Arch, Queen Post

1 INTRODUCTION

The Federal Highway Administration (FHWA), in partnership with the USDA Forest Products Laboratory and the National Park Service, has sponsored a comprehensive national research program on Historic Covered Timber Bridges in the USA [1,2]. The main purpose is to develop improved methods to preserve, rehabilitate, and restore the timber bridge trusses that were developed during the early 1800s, and in many cases are still in service today. The overall goal of the National Historic Covered Bridge Preservation Program is to preserve these iconic bridge structures for future generations. Several studies are under way, while one is aimed specifically 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 recently at three historic covered bridges in the State of Indiana, and four historic covered bridges in the State of Vermont [3,4]. These field testing results will form the basis for more effective 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, and load rating procedures performed to date.

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2 BACKGROUND

Federal regulation requires that like all bridges (greater than 6.1-m span) on public roads, load ratings are periodically determined for historic covered bridges. Often, given the age and complex behavior of these bridges, they are assigned relatively low ratings [5]. It is also widely known that when tested, most bridges are found to perform better than the assigned ratings determined using prudent engineering assumptions. In general, these behaviors 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 geometrical features, a procedure needs to be established on how to safely and reliably conduct load ratings on historic covered timber bridges through physical testing. Furthermore, the developed testing and rating procedure needs to be simplistic and generalized in nature so that practicing engineers have the ability to quickly and accurately analyze and assign safe load capacities to the covered bridges in their inventory with basic, off-the-shelf analysis software.

3 METHODOLOGY

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 two groups of covered bridge with typical truss configurations, namely Burr-Arch and Queen Post trusses. These two truss configurations were selected because they represent a significant population of structures surviving today. The physical load tests provided invaluable structural performance data, from which a recommended procedure for load testing covered bridges is being developed. Additionally, these field tests results are being used to calibrate basic 2D analytical models of the trusses to develop load rating procedures and recommendations. The end result will include recommendations for selection and placement of instrumentation, determination of the maximum experimental load that can be safely applied to the structure, methods of loading the bridge, procedures for ensuring quality data, procedures for analyzing collected data (in conjunction with results from a study on analytical evaluation), and processes for establishing load ratings.

4 BURR-ARCH COVERED BRIDGES - INDIANA

During October 2010, three covered timber bridges consisting of Burr-arch trusses were evaluated and tested in the state of Indiana (Table 1). These bridges were double-arch Burr-arch bridges, and are all located in Parke County, which maintains over 30 historic covered bridges within their roadway network. These single-lane bridge structures are currently restricted to lower weight vehicle loads, but still provide vital transportation links to rural communities in the western part of the state. Approximately 93 covered bridges exist within Indiana.

Table 1: *Bridges tested in the State of Indiana in 2010.*

Name	Built	Length (m)
Portland Mills	1856	36.7
Cox Ford	1913	58.5
Zacke Cox	1908	15.4

4.1 PORTLAND MILLS BRIDGE

The Portland Mills Bridge (County Bridge No. 155) is located on an unpaved road, County Road 650, and allows for vehicular traffic to cross the Little Raccoon Creek as illustrated in Figures 1, 2, and 3. The original structure was built by Henry Wolfe in 1856 and crossed the Big Raccoon Creek in Portland Mills, but was moved to its current location in 1961. In 1991, new transverse glued-laminated timber floor beams and a new roof were installed along with further rehabilitation efforts to bring it back to original condition by 1996.

The bridge is a single-lane, single-span, simply supported double Burr-arch truss. The truss consists of two rectangular parallel bottom chord members with six stop-splayed splice joints per truss, double concentric arches enclosing the truss, one-member upper chords with four stop-splayed splice joints per truss, one-member diagonals and one-member verticals (see Figure 4). The total length of the structure is 36.7 m (120.5 ft.) and is currently posted for a 14.3-t (13-ton) load limit.

Figure 1: *Profile view of the Portland Mills Bridge*

Figure 2: *Interior view of the Portland Mills Bridge*

Figure 3: *Floor system of the Portland Mills Bridge*

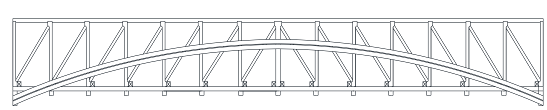

Figure 4: *Elevation view of the Portland Mills Burr-arch truss*

The total height from the bottom of the bottom chord to the top of the top chord was measured to be 5 m (16.7 ft.) and an average truss panel spacing of 2.7 m (8.7 ft.). The connection between structural members, i.e. the connection between the multiple arches and vertical members, is completed by using either a single or series of bolts.

4.2 COX FORD BRIDGE

The Cox Ford Bridge (County Bridge No. 227) is located on Cox Ford Road (unpaved) and allows traffic over the Sugar Creek just west of Turkey Run State Park. Elevation and end view photographs of the bridge are shown in Figures 5, 6, and 7.

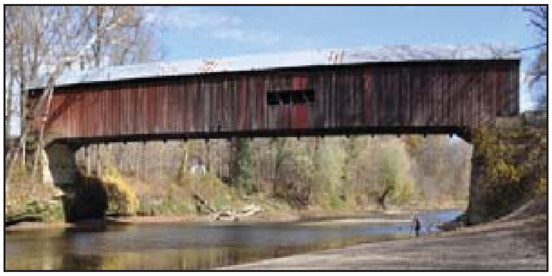

Figure 5: *Profile view of the Cox Ford Bridge*

Figure 6: *Floor system of the Cox Ford Bridge*

Figure 7: *Floor system of the Cox Ford Bridge*

The Cox Ford Bridge was originally built by Joseph A. Britton in 1913 to replace an iron bridge that had been destroyed in a flood. Repairs and rehabilitation completed in 1975 and 1991 included updating the floor beams with glued-laminated timber beams. 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 truss consists of rectangular parallel chords, double concentric arches enclosing the truss, two member bottom chords with nine single-headed hook fishplate and iron shoe splice joints per truss member, one-member upper chord, one-member diagonals and one-member verticals. The trusses and arches are interconnected with iron spikes/bolts at the vertical and diagonal members. The structure is currently rated and posted for a 5.5-t (5-ton) load limit, and the truss configuration is illustrated in Figure 8.

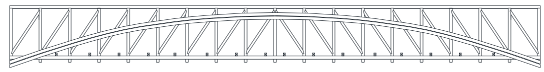

Figure 8: *Elevation view of the Cox Ford Burr-arch truss*

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 an average truss panel spacing of 3 m (9.75 ft.). Just as discussed in the previous bridge, the connections between timber members were accomplished by the utilization of either a single or series of bolts.

4.3 ZACKE COX BRIDGE

The Zacke Cox Bridge is located on Tickbridge Road in Parke County, Indiana, and allows vehicular traffic to cross Rock Run Creek. Elevation and end view photographs of the bridge are shown in Figures 9, 10, and 11.

Figure 9: *Profile view of the Zacke Cox Bridge*

Figure 10: *Interior view of the Zacke Cox Bridge*

Figure 11: Floor system of the Zacke Cox Bridge

The Zacke Cox Bridge was originally built by Joseph A. Britton in 1908 and rehabilitated in 1989, 1991, and 2002. The structure is a one-lane, single-span, simply supported double Burr-arch truss with a total length of 15.4 m (50.4 ft.). The truss consists of rectangular parallel chords, concentric arches enclosing the truss, two member lower chords with single headed hook fishplate and iron shoe splice joints, one-member upper chords, one-member diagonals and one-member verticals and is currently rated and posted for a 14.3-t (13-ton) load limit. The truss configuration for the Zacke Cox Bridge is illustrated in Figure 12.

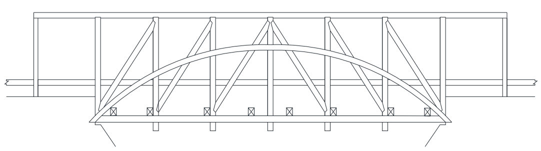

Figure 12: Elevation view of the Zacke Cox Burr-arch truss

The total height from the bottom of the bottom chord to the top of the top chord was measured to be 4.5 m (14.6 ft.) with an average truss panel spacing of 2.6 m (8.4 ft.). All the connections between the timber members, i.e., the connection between the two arches and vertical, were accomplished using either a single or series of bolts.

5 QUEEN POST COVERED BRIDGES - VERMONT

During May 2011, four covered bridges consisting of Queen Post trusses were evaluated in the State of Vermont (Table 2). Two structures were evaluated in Washington and Orange Counties. Two of the bridges are located in town with the other two in rural settings. Approximately 100 covered bridges have survived in the relatively small State of Vermont, which represents the highest concentration of historic covered bridges in the USA.

Table 2: Bridges tested in the State of Vermont in 2011

Name	County	Built	Length (m)
Warren	Washington	1879	17.7
Flint	Orange	1845	27.1
Moxley	Orange	1883	18.6
Slaughterhouse	Washington	1872	20.1

5.1 WARREN BRIDGE

The Warren Bridge is located in the Town of Warren and allows town traffic to cross Mad River to access Hwy 100 and is shown in Figures 13 and 14.

Figure 13: Profile view of the Warren Bridge

Figure 14: Interior view of the Warren Bridge

Figure 15: Floor system of the Warren Bridge

The Warren Bridge was originally built by Walter Bagley in 1879–1880 and is also known as the Lincoln Gap Bridge. The structure is a one-lane, single-span, simply supported Queen Post truss with a total length of 17.7 m (58 ft.). The truss consists of rectangular parallel chords, one-member lower chords, one-member upper chords, one-member diagonals and one-member verticals with no diagonal bracing, struts or counter braces. The structure is currently rated and posted for an 8.8-t (8-ton) load limit. The total height from the bottom of the bottom chord to the top of the top chord was measured to be 4.5 m (14.6 ft.) with an average truss panel spacing of 4.3 m (14 ft.). The truss configuration for the Warren Bridge is illustrated in Figure 16.

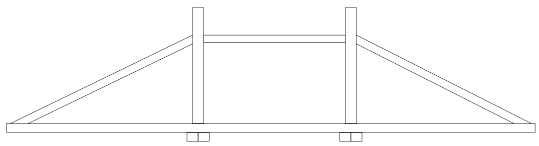

Figure 16: *Elevation view of the Warren Queen-post truss*

5.2 FLINT BRIDGE

The Flint Bridge, shown in Figures 17, 18, and 19, is located north of the Town of Tunbridge, Vermont, and allows vehicular traffic to cross the White River.

Figure 17: *Profile view of the Flint Bridge*

Figure 18: *Interior view of the Flint Bridge*

Figure 19: *Floor system of the Flint Bridge*

The Flint Bridge was originally built in 1845. The structure is a one-lane, single-span, simply supported Queen Post truss with a total length of 27.1 m (89 ft.). The truss consists of rectangular parallel chords, one-member lower chords, one-member upper chords, one-member diagonals, and one-member verticals and is currently rated and posted for a 3.3-t (3-ton) load limit. Aside from the two Queen Posts; all verticals for the Flint Bridge were iron rods, all other members were solid sawn timber. The truss configuration for the Flint Bridge is illustrated in Figure 20. The total height from the bottom of the bottom chord to the top of the top chord was measured to be 4 m (13.3 ft.).

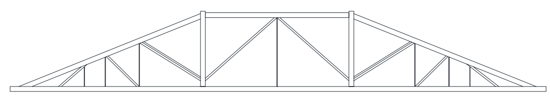

Figure 20: *Elevation view of the Flint Queen Post truss*

5.3 MOXLEY BRIDGE

The Moxley Bridge is located approximately 0.5 mile north of the Flint Bridge near Chelsea, Vermont, and also allows vehicular traffic to cross the White River as shown in Figures 21, 22, and 23.

Figure 21: *Profile view of the Moxley Bridge*

Figure 22: *End view of the Moxley Bridge*

Figure 23: *Floor system of the Moxley Bridge*

The Moxley Bridge was originally built in 1883. The structure is a one-lane, single-span, simply supported Queen Post truss with a total length of 18.6 m (61 ft.). The truss consists of rectangular parallel chords, one-member lower chords, one-member upper chords, one-member diagonals, and one-member verticals and is currently rated and posted for a 4.4-t (4-ton) load limit. The truss configuration for the Moxley Bridge is illustrated in Figure 24. The total height from the bottom of the bottom chord to the top of the top chord was measured to be 4 m (13.3 ft.) with an average truss panel spacing of 4.6 m (15.25 ft.).

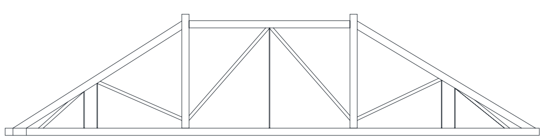

Figure 24: *Elevation view of the Moxley Queen-post truss*

5.4 SLAUGHTERHOUSE BRIDGE

The Slaughterhouse Bridge allows vehicular traffic to cross Dog River near the town of Northfield, Vermont, and is as shown in Figures 25, 26, and 27.

Figure 25: *Profile view of the Slaughterhouse Bridge*

Figure 26: *Interior view of the Slaughterhouse Bridge*

Figure 27: *Floor system of the Slaughterhouse Bridge*

The Slaughterhouse Bridge was originally built in 1872. The structure is a one-lane, single-span, simply supported Queen Post truss with a total length of 20.1 m (66 ft.). The truss consists of rectangular parallel chords, one-member lower chords, one-member upper chords, one-member diagonals, and one-member verticals and is currently rated and posted for an 8.8-t (8-ton) load limit. The truss configuration for the Slaughterhouse Bridge is illustrated in Figure 28. The total height from the bottom of the bottom chord to the top of the top chord was measured to be 3.7 m (12.25 ft.) with an average truss panel spacing of 3.7 m (12.25 ft.).

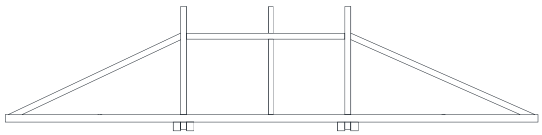

Figure 28: *Elevation view of the Slaughterhouse Queen Post truss*

6 CASE STUDY: FLINT BRIDGE, VERMONT

Currently, analytical modeling and analysis of field test results is being completed on both the Indiana and Vermont bridges for investigation of improved load testing and load rating techniques for these types of covered timber bridges. The following is a case study focusing on the preliminary field test and analytical modeling results from the Flint Bridge in Vermont. The procedures outlined for this case study will be similar for all the subject bridges although the results will vary slightly because of differences in bridge geometry, member connectivity, deterioration, and other factors.

6.1 FIELD TESTING

Testing of the covered bridges involved installing displacement and strain transducers on the structures 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 three locations on each truss: 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 software. Figure 29 illustrates a typical setup for the measurement of global deflection.

Figure 29: *Typical instrumentation setup for covered timber bridge global deflection measurement*

Member strains were recorded at various locations on one 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 was an issue, smaller diameter drywall screws were used to minimize the holes left upon removal of the strain gage. Because of the limited number of gages available and time constraints, symmetry was assumed on the trusses and only one truss was instrumented for strain measurement.

Loading for the Flint Bridge was a Ford F-450 flatbed pickup, shown in Figure 30. The total vehicle weight of the F-450 was 4,726 kg (10,420 lb.), rear axle weight was 2,595 kg (5,720 lb.), and the front axle weight was 2,132 kg (4,700 lb.), as illustrated in Figure 31. The axle spacing for the truck was 4.5 m (14.6 ft.), and the wheel base was 1.9 m (6.25 ft.). Because of the limited roadway width, only one load case was investigated, and involved the load truck straddling the center line along the bridge length. For all runs the load truck travels across the bridge at a crawl speed, or approximately 4.8–8 km/h (3–5 mph) and travels from East to West.

Figure 30: *Ford F-450 flatbed truck used for load testing the Flint Bridge*

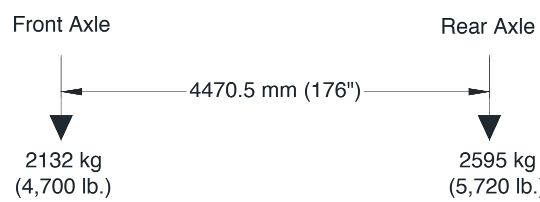

Figure 31: *Flint Bridge load truck axle weights and dimensions*

6.1.1 Deflection Results

Illustrated in Figure 32 is the displacement of the midspan of both trusses versus truck position. The data indicate that for the Flint Bridge, the displacement of the midspan of the trusses does not initiate until the front axle of the truck is nearly one-third the way across the span. This corresponds to the approximate location of the first bottom chord splice. In addition, the end bearing for this structure consists of a corbel that is approximately 12 ft. long with the interior edge of bearing being nearly 6 ft. from the end of the arch. The large corbel, long-bearing length, and location of the bottom chord splice result in negligible movement of the midspan of the truss until the load truck reaches the bottom chord splice located at approximately the third point of the span.

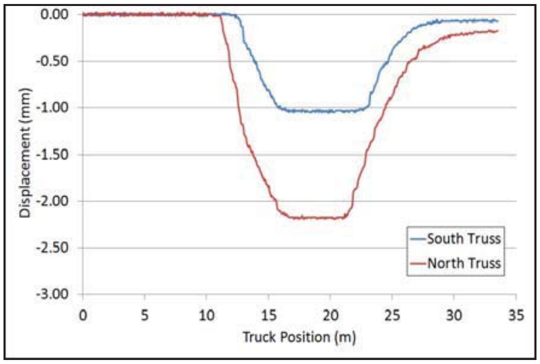

Figure 32: *Midspan displacement plot for the bottom chord of the Flint Bridge during live load testing*

6.1.2 Strain Results

Illustrated in Figure 33 are the locations of strain gages on the Flint Bridge.

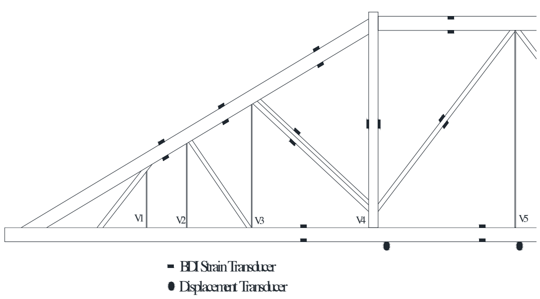

Figure 33: *Typical strain gage placement for the Flint Bridge, east half of south truss (gages mirrored on west end of truss)*

Illustrated in Figures 34, 35, and 36 are the typical strain plots from field testing the Flint Bridge for the bottom chord, vertical, and diagonal, respectively. Inspection of the bottom chord strains in Figure 34 suggests that initially the bottom chords experience relatively small strains, likely axial strain, but as the load truck approaches and crosses the location of the bottom chord splice, the strain response turns to bending and the magnitude of strains greatly increases. This pure bending behavior encourages the use of beam elements for the bottom chord members in the analytical model discussed in the next section.

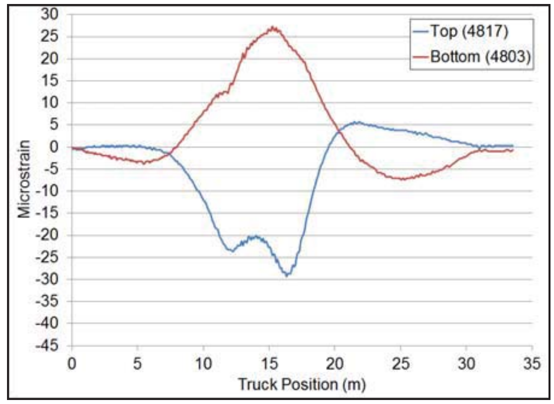

Figure 34: *Typical strain plot for the bottom chord of the Flint Bridge during live load testing*

Inspection of the strains in Figure 35 also hint at some amount of bending in the vertical members, but for the most part the member is in tension as would be expected. Last, examining the strain plot for the diagonals of the Flint Bridge in Figure 36 indicates an initial state of tension in the members and then as the load truck approaches the middle of the span going into a state of compression for the remainder of the test.

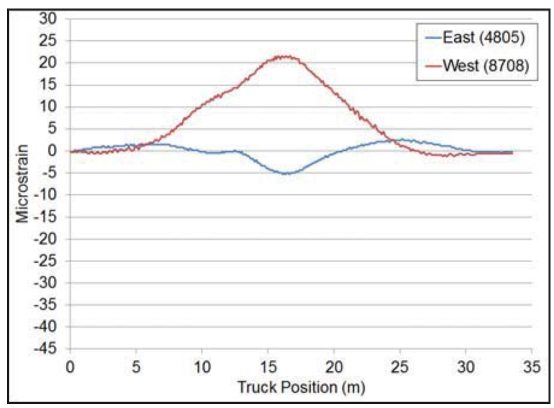

Figure 35: *Typical strain plot for the verticals of the Flint Bridge during live load testing*

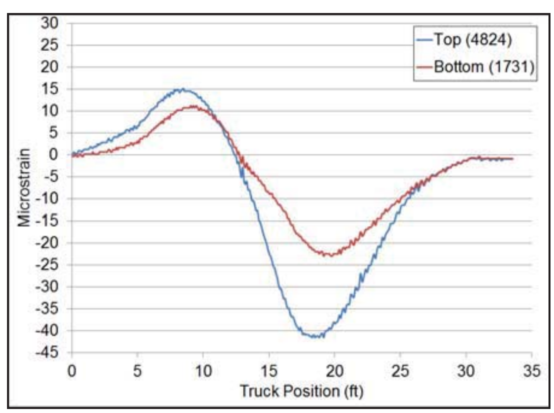

Figure 36: *Typical strain plot for the diagonals of the Flint Bridge during live load testing*

6.2 ANALYTICAL MODELING

Because of its familiarity and widespread use in the engineering community, we selected STAAD (Bentley, Exton, Pennsylvania) finite element modeling software for this work. 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. Bridge dimensions and member sizes were obtained from field inspection notes taken during load testing and/or as-built plan sheets obtained from the owner. Illustrated in Figure 37 is the 2-D STAAD model of the Flint Bridge.

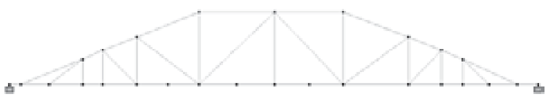

Figure 37: *STAAD model of the Flint Bridge*

Calibration of the model was carried out by applying the load truck used in the field tests and comparing the analytical results with the results from the field testing. Point loads that simulated the load truck were applied to the structure in 30.5-mm (0.1-ft.) increments across the structure to simulate a moving load truck as used in the field testing. As the bridge was only being modeled in 2-D and only one truss was being modeled, transverse distribution was assumed to be 50 percent such that one wheel line went to each truss. Field test data verified this method of transverse distribution to be accurate for this bridge. Figure 38 illustrates the point loads used in the STAAD analysis for calibration of the model with field test data.

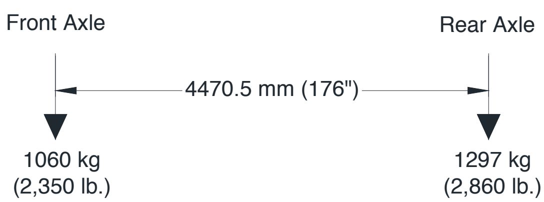

Figure 38: *Flint Bridge load truck axle weights for 2-D modeling in STAAD*

Initial results from running the analysis on the pinned-pinned model were found to be inaccurate; therefore, the bridge model was adjusted to fixed-fixed in an attempt to bound the results. Results 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. Next, 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. The ultimate goal is to make the modeling process as straight-forward as possible.

Subsequently, through trial and error, a 2-D model of the truss was developed that improved the accuracy of the results. The current 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 70–80 percent. Further improvements in the results from the STAAD model are optimistic with other minor modifications to the model as the project progresses.

Although the wall sheathing and roof of these covered bridges are not intended to be structural elements, they likely contribute to the stiffness of the structure in some regard. Note that these and other unknown factors contributing to the performance of the actual structure will not be represented in the STAAD model. Furthermore, they may be partially to blame for the discrepancy between the field results and the analytical modeling results presented herein. These factors are still being evaluated and will be addressed in the final manual developed from this work. Final recommendations will address these characteristics and point out potential methods for handling their influence on the bridge performance.

6.3 LOAD RATING

The final step in the process is performing a load rating for the subject bridges using the calibrated finite element (FE) model developed for that structure from live load testing results. The basic procedure for performing the load rating via the calibrated FE model is as follows: 1) based on the findings and recommendations from the previous section, create a FE model of the structure; 2) run rating vehicles and/or different 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); 3) calculate the capacity of each key member to be evaluated taking into consideration any deterioration or decay found during inspection; 4) calculate the ratio of the member capacity to the member forces output from the FE model to determine the load rating factor. A load rating factor greater than or equal to 1 is desired.

7 CONCLUSIONS

Currently no formal load rating procedures exist for historic covered timber bridges based on load tests, and given their complex design and age they are often assigned conservative ratings. Additionally, previous research has determined that performing live load tests on bridges often results in higher load capacities compared with load ratings based upon engineering assumptions.

To date, two different designs of covered timber bridges have been load tested. These include three Burr-arch covered bridges in the state of Indiana and four Queen Post covered bridges in the state of Vermont. The load tests conducted on these structures involved the installation of displacement and strain transducers on and inside the structure, loading of the structure with a vehicle of known weight, and collecting the data for analysis. The measured displacements and strains are and will further be used to calibrate finite element (FE) models such that the models accurately simulate the performance of the structures under the same vehicular loads. The significance of this calibration process is two-fold: 1) once the models are calibrated, and assuming that the members stay elastic, the models may be used to analyze the performance of the structures for any given vehicle, and thus perform load rating on a bridge; 2) based on the collected data and feedback from the FE model, develop standard procedures for sensor type and placement to be used on similar structures in the future as well as recommendations for loading type and configuration.

The final product will be recommendations and guidelines for instrumenting covered bridges, load testing covered bridges, generating a simple but accurate bridge model of covered bridges, and load rating covered bridges such that load limits that are both safe and reflective of the actual performance of the structure may be assigned to the bridge.

8 CONTINUED/FUTURE WORK

Field testing will continue in the spring/summer of 2012 with the testing of a group of Howe truss-covered timber bridges. In addition, analysis of already collected field test data continues with emphasis on the behavior of the key members: top and bottom chords, diagonals, verticals, and for the Burr-arch bridges the arches such that FE model calibration and load rating procedures can be finalized.

Once final FE models and load ratings for the subject bridge have been completed, procedures and guidelines for instrumenting, loading, load testing, FE modelling, and load rating similar covered timber bridges will be documented and published in a format similar to load rating guides for other bridge types.

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