Laboratory tests of a timber trestle bridge chord
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ABSTRACT
A full-scale specimen replicating the main elements of a three span timber trestle bridge chord was constructed then disassembled and reassembled into a two span, and then into a one span, specimen. Laboratory load testing was conducted to examine the service load behavior of the specimens. Selected results for several loadings are presented. Comparisons are made between the laboratory result, some elementary analytical beam models, and a rigorous finite element model. The models exhibited improved accuracy as rigor was added. Relative displacement between components is a significant factor in and challenge in modeling the specimens.

INTRODUCTION
In 1995, researchers from Colorado State University (CSU) and the Association of American Railroads (AAR) jointly conducted extensive field load tests of three open deck, timber trestle railroad bridges (Gutkowski, et.al., 1998, 1999). Numerous static and moving train loads as well as ramp and sinusoidal actuator loads were applied. The main motivation for the testing was to examine the load sharing aspects within the chords. The outcome of the field tests have been published (Gutkowski, et.al. 1997, 1998, 1999) and in a M.S. thesis (Robinson, 1998). As train loads have been increasing significantly in recent decades, required axle loads are due to be increased by 30% in the AREA railroad bridge code (AREA, 1995). So a possible need to strengthen old timber trestle bridges was a concern to the AAR. Later in 1995, one of the tested bridges (Bridge 101 in Pueblo, Colorado) was strengthened by the addition of one stringer ply (sleeper beam) in each chord. The bridge was retested in spring 1996. Useful findings resulted, but were site specific (Gutkowski et al. 1999). Consequently, to conform to more idealized (design basis) behavior, full-scale tests were conducted in the laboratory. The laboratory specimen was nearly a replication of the field bridge geometry. Some aspects of the comparison between the field study and the lab specimen have been reported (Gutkowski, et.al. 1999) and those an other findings will be published in an M. S. thesis under preparation by the second author (Doyle, in preparation). This paper presents the current findings of the study.

STANDARD OPEN DECK TIMBER TRESTLE BRIDGE CONFIGURATION
A predominant number of existing timber railroad bridges in the U.S. are standard open deck, timber trestle bridges. Fig. 1 is a schematic illustration of such a bridge. A longitudinal "chord" consisting of a 3 to 5 "plies of stringers" (4 are shown) is centered below each steel rail of the track. The two rails are supported by wood cross-ties spanning the chords. Ends of interior span chords are supported by pile bents, comprised of a solid sawn timber cap and round wood piles. End piers are similar and have a timber retaining wall. Plies of the chords are either "packed" tightly together side by side or "spaced" with gaps of about 1-4 inches between adjacent plies. In some cases, steel tie rods are used to laterally connect the plies at selected locations. Details of the design and construction of these bridges are available in the applicable specifications code for railroad bridges.

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As noted above, in 1995, a CSU-AAR research team tested three open deck timber bridges in the field. Bridge No. 101 was one of the bridges tested. The main laboratory specimen used in the research described herein was a three-span chord configured to be similar to Bridge No. 101. The actual bridge is a right bridge and has 3 spans of 4.26 m, 4.59 m and 4.26 m. The bridge supports a slightly curved rail. Main components are creosote treated Douglas fir timbers. Before the retrofit, each chord had four packed plies of stringers in a staggered, two span continuous pattern. In the end spans, alternate plies were simply supported. Plies of stringers were tied together by steel rods located near the caps. A plank walkway existed on each side. The caps were solid sawn timbers supported by five round piles. Caps were lag screwed to the piles and the chords were through-bolted to the caps. Piles were X-braced by two wood members bolted to each pile.

In 1996, Bridge 101 was strengthened by the addition of a stringer ply to each chord. Added plies were the same size as the existing plies. They were installed by force fitting them between the existing cross-ties and pier caps. They were placed so as to be consistent with the existing lapped pattern of one and two span plies. The plies of the chord were not recentered under the steel rail. This is preferred by some railroad owners as it make the retrofit easier. Lateral steel tie rods were removed and replaced with longer ones.

LABORATORY SPECIMENS

Each rail of a track carries a line of train wheels. In a right bridge each chord carries essentially the same side by side load pattern. Due to variability of materials, a slight difference in resistance of each chord is exhibited and the two wheels of any axle can transfer slightly different loads, but a right bridge is essentially symmetrically loaded about its centerline. This symmetry allowed the researchers to economize on materials. Specifically each laboratory test specimen was only one chord of bridge plus other associated structural components.

Specimen 1 was a one span (4.37 m) chord consisting of five plies. Each ply was about 203 mm x 400 mm and supported by 347 mm high x 294 mm deep caps. The caps were supported two short round pole stubs (about 305 mm diameter and 330 mm high) atop a concrete pad bolted to the strong floor. Pieces of timber railroad ties were placed about 70 mm apart over the entire span. Later, a retest was done after centering the rail over four plies. Then, one outer ply was removed and the 4 ply specimen was retested.

Specimen 2 was similar to specimen 1, except it was a two span (each 4.37 m) chord and the four plies were all two span continuous. At mid span, the cap was supported as in specimen 1. At the ends the cap was atop short round stubs 51 mm high supported on steel plates on large concrete filled barrels placed on the floor.

Specimen 3 was a three span continuous chord. Each span was 4.37 m and the plies a combination of single span and two span plies laid out in the pattern shown in Fig. 2. Interior supports were made as in specimen 1. The end supports were the same as in specimen 2. After testing the specimen, a single span ply was added to the outside of each span. A load test was conducted without adjusting the rail position (i.e. it was left "off center").
The three specimens were made from a common set of main materials. Specimens 3, 2 and 1 were assembled in that order. In each specimen, the ties were not spiked to the stringers and the ends of the stringers were not bolted to the caps. The steel rail was placed over the full length but only tied to the plies locally near the ends of the span. These aspects were necessary because the specimens had to be readily disassembled in order to reuse the materials in the other specimens. Presently, the three span specimen has been reconstructed and will be fully interconnected for conduct of tests for comparison.

Additional load tests were made on each specimen after either purposely misaligning plies, creating differential end support levels, or removing steel tie rods, etc. to examine the effects of such field irregularities that occur over time.

MATERIAL PROPERTIES

For the laboratory specimens, the MOE values of the plies of stringers were determined by actual load tests of each ply. Loadings used were consistent with those studied in the laboratory and models of the bridge chords, themselves. This was done to include shear deformation in a consistent manner for laboratory tests and analytical modeling. Doyle (Doyle. in preparation) provides the individual ply MOE values for each span.

LOADING PROCEDURES

Point loads were applied using pair of hydraulic actuators, each attached to a movable overhead steel frame. Either a single point load or a pair of point loads was then applied, depending on the particular specimen. The two loadings for Specimen 1 were: 1) a single point load was applied at mid-span, and 2) pair of point loads (2.76 m apart), centered longitudinally about mid span. For specimen 2, a single point load was applied at midspan of one of the end spans. A pair of point loads (2.76 m apart), centered longitudinally about midspan, was later applied to the same end span. For Specimen 3 a single point load was applied at midspan of the middle span. A pair of point loads (2.76 m apart), centered longitudinally about midspan, was applied in the middle span. For the single point load, the actuator was incrementally ramped to approximately 306 kN. For the two-point loading both actuators were simultaneously incrementally ramped to that load level.

INSTRUMENTATION

Potentiometers were used to measure vertical displacement relative to the ground at mid-span and quarter points of each span. All plies were monitored for each load configuration. Thirty-six (Forty-five) deflections were monitored for the three-span, 4 ply (5 ply) specimen. For the two-span specimen 24 positions were recorded. For the one-span 4 ply (5 ply) specimen 12 positions (15 positions) were monitored. These are detailed in a thesis on the work (Doyle. in preparation).

EXPERIMENTAL RESULTS

Extensive data was taken from the various load tests. Doyle's thesis will provide detailed results and findings. Selected results are shown in this brief overview paper. Figure 3 shows 4 cycles of loading and unloading of one ply in the loaded span specimen 2 under a point load applied at midspan of one span. Except for initial motion at low load level, the linearity of the load-deflection response is evident. This was typical of the response at other points in the loaded span of each specimen. The corresponding individual ply displacements for the two span specimen at the 306 kN load level are shown in Fig. 4.
In the end spans of Specimens 1 and 2, which had no load on them, uplift was evident in the response. This reflects the absence of connectivity of the ply ends to the support, in these initial tests. Measurements were not taken directly at the supports. But it is evident in Fig. 4 that some downward movement occurred at the two supports of the loaded span. This is addressed subsequently. It also appears that the ply ends moved off upward and off the end support, at the exterior end of the span without load on it. Similar support behavior occurred in the other specimens. These are discussed subsequently.

**ANALYTICAL MODELING**

Several numerical modeling methods were utilized to predict the deflections of the experimentally investigated timber bridge chords. In an earlier work, ordinary beam models (one-, two-, and 3- span continuous beams) were applied to the specimens. The steel rail was not included because it was only fastened nominally at a few locations and was believed to not transfer significant interlayer shear. For the one span bridge chord, the multiple ply chord was modeled as a single span, simply supported beam. As load sharing among plies was neither known nor directly measured, the plies were assembled into a single member, having an EI value equal to the summed I values of the plies times their average MOE. For the two-span (three span) specimen, the same assembly was done, except the resulting beam was a two-span (three-span) continuous member. Except for the single span case, these models did not work well, giving low deflections.
For Specimen 3, a "semi-continuous" beam model was later employed. This three span chord specimen had lapped two-span plies in the middle span and in-filled single span plies in the end spans (see Fig. 2). The semi-continuous beam model used a stiffness matrix assembled for the system schematically depicted in Fig. 5. Referring to that figure, over the length of the specimen a line of plies has either a configuration A (i.e. that of the upper beam with a hinge at the third support), or a configuration B (i.e. that of the lower beam with a hinge at the second support) of the schematic. Thus a stiffness model is made for each configuration, with the individual spans assigned an EI equal to the sum of the individual EIs of the plies that have the configuration of that line. The rigid link at each midspan is to force the each span of the two configurations to be joined in resisting the single applied load. The hinges are used to reflect the discontinuity of the butted ends. The steel rail was neglected. It also gave low predicted values.

Fig. 5. Three Span Semi-Continuous Beam Model

The error in the above models was primarily attributed to support motion and similar relative displacements due to gaps between other surfaces. It was also understood that the steel rail contributed some resistance, at least that due to its centroidal moment of inertia as an offsetting factor. Consequently, modified beam models were later employed. For Specimen 2, a layered beam was used which included the steel rail and free end at the exterior support of the span without load on it. The layered beam was created as a planar frame, with the steel rail linked to the ply members by short vertical links. These links corresponded the locations where the steel rail was actually connected to the plies. The discontinuous locations in the steel rail and the plies of stringers were incorporated as internal hinges. For Specimen 3, the same type model was used and the exterior supports of both end spans were free to move vertically.

The above analytical effort was done to determine if elementary type models worked for practical application. In addition, exploratory advanced modeling was done by a three-dimensional finite element model that was used to simulate the behavior of the specimens for research purposes. The finite element model was built using the commercial AxisVM structural modeling software (Inter-CAD Ltd. 1999). Beam elements were used to model the plies, railroad ties and steel rail. Gap elements were used to model support conditions so as to allow uplift and take into account initial gaps that exist between the plies and the supports due to unequal ply heights and imprecise overlapping of the alternate two and one span plies (see Fig 2) resulting in clearance at the butt joints. The initial gaps were determined based on the average of the measured displacement values at nearby measurement points. For each ply member, a physically measured MOE was used in the analysis. To date, four models have been developed corresponding to the one span with five plies; two span with four plies; and three span with four and five plies specimens.

**COMPARISON OF ANALYTICAL AND EXPERIMENTAL RESULTS**

Figure 6 shows the measured and calculated results for the middle span of the two-span, four ply chord specimen under a midspan point load in the end span. The actual measured displacements are shown in triangles with a fourth order regression curve fit imposed. This plot includes the support motions, downward (upward) near the interior (exterior) supports. As the ordinary beam models did not include support motion, this data set (plot) was graphically translated and rotated in space so as to sit the plot atop the interior supports, i.e. to remove the interior support movements. This plot is shown in circles. The results of the modified beam model are shown in diamonds (Xs) for the case of the vertical links being planar truss (planar frame) members hinged (rigidly connected) at their ends to the plies and steel rail. The resulting midspan deflection was .3068 cm (.7844 cm) for the hinged (rigid) link model. If the average support motion is added the corresponding result is .9088 cm for the hinged link model. The corresponding result for the finite element model was 0.9732 cm (average of the 4 plies). The corresponding measured value was 1.0325 cm. (average of the four plies). These results are listed in Table 1. Table 1 also lists midspan displacements (measured and predicted) of the loaded span for three other specimens for the one point load cases.
Fig. 6. Chord 2/4 – 1 Point Load, Deflection Profile for ‘Ideal 2 Span Bridge Chord’

Table 1. Comparison of Measured vs. Predicted Deflections at Loaded Point a,b

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Ply Orientation</th>
<th>Measured (cm)</th>
<th>Hinged Beam Model (cm)</th>
<th>FE Model (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (one-span)</td>
<td>Far Outside</td>
<td>-1.3736</td>
<td>-1.5220</td>
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<tr>
<td></td>
<td>Far Inside</td>
<td>-1.3053</td>
<td>-1.2934</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Near Inside</td>
<td>-0.9401</td>
<td>-1.0344</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Near Outside</td>
<td>-0.7452</td>
<td>-0.7450</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5th Added Ply</td>
<td>-0.6132</td>
<td>-0.4499</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>-0.9954</td>
<td>-1.1030</td>
<td>-1.0089</td>
</tr>
<tr>
<td>2 (two-span)</td>
<td>Far Outside</td>
<td>-1.1102</td>
<td>-1.0374c</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Far Inside</td>
<td>-1.0152</td>
<td>-1.0094c</td>
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</tr>
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<td></td>
<td>Near Inside</td>
<td>-0.9705</td>
<td>-0.9590d</td>
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<tr>
<td></td>
<td>Near Outside</td>
<td>-1.0325</td>
<td>-0.8870d</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>-1.0323</td>
<td>-0.9088c</td>
<td>-0.9732d</td>
</tr>
<tr>
<td>3 (three-span)</td>
<td>Far Outside</td>
<td>-1.0109</td>
<td>-1.1777c</td>
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</tr>
<tr>
<td></td>
<td>Far Inside</td>
<td>-0.9474</td>
<td>-1.1298d</td>
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</tr>
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<td>Near Inside</td>
<td>-1.0348</td>
<td>-1.0592c</td>
<td></td>
</tr>
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<td></td>
<td>Near Outside</td>
<td>-1.0526</td>
<td>-0.9669d</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>-1.0114</td>
<td>-0.9238d</td>
<td>-1.0834d</td>
</tr>
<tr>
<td>4 (three-span)</td>
<td>Far Outside</td>
<td>-1.0660</td>
<td>-1.2349c</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Far Inside</td>
<td>-0.7912</td>
<td>-1.1066c</td>
<td></td>
</tr>
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<td></td>
<td>Near Inside</td>
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<td>-0.9426c</td>
<td></td>
</tr>
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<td></td>
<td>Near Outside</td>
<td>-0.8684</td>
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<tr>
<td></td>
<td>5th Added Ply</td>
<td>-0.6269</td>
<td>-0.5763c</td>
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</tr>
<tr>
<td></td>
<td>Average</td>
<td>-0.8565</td>
<td>N/A</td>
<td>-0.9231c</td>
</tr>
</tbody>
</table>

a. Load was 306000±667 N at midspan
b. Loaded span was the single (end, middle) span for specimen 1 (2, 3, 4)
c. A 3.500 mm initial gap at the middle support was accounted for.
d. A 3.734 mm and 3.048 mm initial gaps at the left and right supports of the loaded span were accounted for.
e. A 3.302 mm and 2.794 mm initial gaps at the left and right supports of the loaded span were accounted for.
Initial results show only modest differences between the experimental results for the 1-span, 2-span and 3-span specimens. As expected, the 3-span, 5-ply specimen measured deflection was reasonably close to being 80% (actually was 85%) of the deflection of the 3-span, 4-ply specimen. The results listed in Table 1. Loose fit of the added ply contributed to the higher outcome. For the average deflection of the plies of the one span specimen, the predicted results were not close for the FE model vs. the hinged beam model (the error is +1.8 % for the FE, and +10.8 % for the beam model with support movement included). For the 2-span (3 span), 4-ply specimen the errors were -5.7% (+7.1%) for the FE and -12.9% (-8.7%) for the hinged beam model. For the 3-span, 4-ply specimen, the error in the FE result was +7.8%. No value for the hinged beam model is yet available. Generally, the more rigorous FE model provided closer predictions of the measured behavior. Some of the differences between the analytical results and measured values are due to the inclusion of shear deformation in the tests for MOE values and averaging the deflection results to a constant value in the various beam models. These aspects will be refined in later studies. The three-dimensional FE model shows a slightly different distribution of the displacements between individual plies than the measurements, and the measured ply MOE values varied significantly, too. Due to a large number of different beam elements in the FE model, the results tend to reflect the effect of a mean MOE value. In the beam models an averaged MOE was used.

CONCLUSIONS

Considering the complexity of the physical specimen and its systems behavior, very good agreement between the measured and predicted deflections resulted. Initial gaps between the plies and the supports were taken into account in the FE. These results support the suspicion that gaps are a significant factor, and additional tests are planned to extensively monitor and examine that influence. The hinged link beam model gave better predictions than the rigid link beam model. Due to the direct influence of the MOE and of the initial gaps on displacements and the extensive presence of gaps throughout the specimen (e.g. between ties and plies) a probabilistic approach will be included in later load tests and analytical studies.

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REFERENCES


Doyle, K. in preparation. Laboratory tests and analysis of full-scale timber trestle railroad bridge chords, M.S. Thesis, Department of Civil Engineering, Colorado State University, Ft. Collins, CO.


