Field load testing and modeling of a strengthened timber trestle railroad bridge

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Abstract

In the United States many existing timber trestle railroad bridges have been in service for between 40 to over 100 years. During that time period, train loads have increased significantly. This paper describes the load tests of such a bridge before and after it was strengthened and selected results and findings. Static, ramp, and rolling train loads were used before the strengthening and rolling train tests were used afterward. The goal is to better understand load paths, examine behavior under current load levels, and examine the benefits of strengthening. The objective is to reduce the extent of strengthening needs in the national inventory.

1 Introduction

In the summer of 1995, researchers from Colorado State University (CSU) conducted field testing of three open deck, timber trestle railroad bridges [1, 2]. The Association of American Railroads (AAR) collaborated in the work. In the fall of 1996, one of the bridges was strengthened by the addition of one stringer ply (sleeper beam) in each chord. The bridge was retested in the spring of 1997.
2 Open deck timber trestle bridge configuration

Figure 1 depicts a standard open deck, timber trestle railroad bridge. A chord is centered below each rail of the track. Each chord consists of multiple plies of timber stringers. Wood cross-ties placed across the chords support the steel rails. The intermediate supports are pile bents, comprised of round wood piles and a solid sawn timber cap. End abutments are similar and normally have a timber retaining wall. The plies of the chords are either "spaced" with gaps of a 25-102 mm between adjacent plies or "packed" tightly together.

3 Selected bridge sites

The field tested bridges were identified by a railroad numbering system. Bridge No. 32.35 was a right bridge approximately 152 m long (31-spans at 4.92 m) with spaced plies of stringer. Bridge No. 32.56 was a 4 span bridge approximately 22.6 m long. End abutments were perpendicular to the track and bridge centerline but intermediate bents were skewed at about 30 degrees. Bridge 101, the focus of this paper, is described in detail below.

3.1 Bridge No. 101 (Pueblo, Colorado)

This 3 span bridge was about 13.1 m long, with spans of 4.26, 4.59 and 4.26 m. A plank walkway existed on each side. Minor repairs had been made including shims, seals, plates on caps, several pile replacements, straps between piles and stringers, and replacement of ties.

Main components were creosote treated Douglas fir timbers. Each chord had four packed stringers (165 mm wide and 394 mm deep) in a staggered, two-span continuous pattern. In the end spans, alternate plies were simply supported. Plies were tied together by steel rods near the caps only. The bridge supported a slightly curved rail on 2.95 m long ties, 222 mm wide by 216 mm deep.

The caps were solid sawn timbers 343 mm wide by 387 mm deep, 4.59 m in
length. Each cap was supported by five round piles, each about 305 mm in diameter. The exposed length of the piles ranged from .33 to 1.31 m. Caps were lag screwed to the piles and the chords were through-bolted to the caps. Piles were X-braced by two 102 mm by 203 mm wood members bolted to each pile.

In November 1996, the bridge was strengthened by the addition of a ply to each chord. The plies were of the same size as the existing plies and consistent with the existing staggered pattern of one and two span plies. The retrofit required the removal of the cantilevered walkway. Because the plies of a chord were not recentered under the steel rail, the ties and steel rails did not have to be removed. These made the retrofit easier, but required the new ply to be forced into place between the bottom of the ties and the caps. Lateral bolts were removed and replaced with longer connectors to allow for the additional ply. In March 1997, the bridge was tested under controlled rolling train loads at various speeds.

4 Material Properties

4.1 Non-Destructive Evaluation

Extensive field measurements were made of the modulus of elasticity (MOE) and modulus of rupture values (MOR) values for all members. An ultrasonics-based, non-destructive evaluation technique was employed using an instrument developed in Switzerland [4].

4.2 Results of Property Measurements

MOE values were measured at 24 stringer ply locations, 8 cap locations and 12 pile locations. Data was taken in December, 1995 with temperatures ranging from 2 to 9 degrees C. Stringer ply values ranged between 11,000-17,000 MPa. The mean was 14,200 MPa with a COV = .124. Cap values ranged between 6,610-13,200 MPa. The mean was 10,600 MPa with a COV = .208. Pile values ranged between 8,550-15,200 MPa. The mean was 12,100 MPa with a COV = .152. MOR values were directly obtained for 162 locations. Stringer ply values ranged between 10,000-24,000 kPa (mean = 20,000 kPa , COV = 0.221). Cap values ranged from 10,000-24,000 kPa (mean = 19,300 kPa, COV = .213). Pile values ranged from 8,000-24,000 kPa (mean = 14,900 kPa (2155 psi), COV = 0.395). Moisture content (from the instrument) was between 9.6-10.5%.

5 Loading Procedures

5.1 Track Loading Vehicle

The AAR's Track Loading Vehicle (TLV) was used to apply static load and ramp loads (as described below) in the initial test. The TLV is a specialized train car
fitted with hydraulic loading capability. Rolling train loads were also conducted. Exploratory dynamic excitation (sinusoidal) load testing was also done by shaking the TLV, but is not described in this paper.

5.2 Static and Ramp Loading

Fig. 2 depicts the three car test train used in the initial tests. The TLV was accompanied by its instrumentation car (IC) and a locomotive. Various multi-point static loadings were achieved by positioning the twelve axle (actuator bogey having been lifted off the track) train at different locations. Axle loads for Bridge 101 are listed in references 1-3,6. The total static weight used in the initial tests was 3010 KN.

Sequentially, the train was moved along and positioned by radio command until a selected axle was within +/- 25 mm of a marked location on the track. “Positioning” implies having specified at what point along the bridge an axle was to be placed to achieve the desired position of the overall train. In some cases, two axles were centered about a desired point (for example, about a pier or about mid point of a span). This was readily done and any error was recorded within 13 mm. The number of load positions used in several passes was 108.

For ramp loading, two actuators lift the TLV at mid-point and to transfer part of its weight to the bridge. Ramp loads investigated the response up to present American Railway Engineering Association code limits on axle loads and beyond to potential new code limits. This was done at 42 locations. The bridge sustained the ramp loads and responded linearly, except for some initial take up.

Fig. 2 Configuration of test train

5.3 Rolling train loads

In the initial testing, pilot rolling train load tests were conducted to assess the potential methods of assessing dynamic response (1, 2, 5). Speeds were less than 16 Km/hr due to limitations set by the specific railroad owners. In the test of the strengthened Bridge 101, a four car train was used without the TLV and IC. The first three cars had a total static weight of 3500 KN and moderately different axle spacings. The fourth car weighed 1220 KN. Only rolling train loads were done, but at speeds from 3.2 Km/hr to 32 Km/hr.
6 Instrumentation

6.1 Static and Ramp Load Tests

Displacement transducers (LVDTs and potentiometers) were used to measure absolute (relative to the ground) vertical displacement and relative (between adjacent bridge components) vertical displacement, primarily between ply ends and cap and between cap and piles). The former reference was to measure absolute motion. The latter reference was to eliminate ply end support motions, so as to be able to compare with idealized models that do not include them. Instrumentation locations varied from a few selected plies in several spans to all plies in all spans being instrumented. Extensometers were used to measure longitudinal deformation in selected ply members and piles. Instrumentation was repositioned for different sequences of loading. Electronic instrumentation was partially backed up with optical surveying measurements.

6.2 Rolling train loads

In the initial pilot tests, timing and triggering of data acquisition were done by stop watch and manual action. Instrumentation was the same as in the static/ramp load tests.

In the retest of Bridge 101, potentiometers were used at 72 ply locations. This enabled simultaneous measurements of vertical motion (all relative to the ground) in all plies of both middle span chords and of the two end span chords on one side of the bridge. Measurements were taken at midspan and adjacent to the supports plus at some selected locations in other plies. Inductive triggers were used to electronically turn data acquisition control on and off, thereby capturing the response for specified locations of the moving train.

7 Results

A comprehensive report [1] and a M.S. thesis [4] were written on the initial load tests on the three bridges. A full report on the retest of Bridge 101 is in preparation. A few sample results for Bridge 101 are presented.

7.1 Static Load and Ramp Loading (Initial Tests)

Figure 3 depicts midspan displacement measurements in the end spans for the TLV test train positioned with the front axles of the locomotive in Span 3 and its lead back axle in Span 1. Selected plies were monitored (relative to the ground) in each chord and some variation in the displacements is evident.
To assess load sharing, the deflection values were adjusted empirically. Plies share load somewhat in proportion to the MOE values. Thus, each displacement was multiplied by its MOE value. Each ply is a simply supported member, so idealized support condition is not a distinguishing factor. Adjusting for MOE, the values for the east plies 1 and 3 are \(0.00546 \text{ m} \times 11,870 \text{ MPa} = 64.8 \text{ MN} \) and \(0.00559 \text{ m} \times 16,550 = 92.5 \text{ MN/m}\), respectively. Evidently, interior ply 3 carried a larger load share. The adjusted values for the west plies 5 and 7 are \(0.00512 \text{ m} \times 14,100 \text{ MPa} = 72.5 \text{ MN/m}\) and \(0.0064 \text{ m} \times 12,800 \text{ MPa} = 82.1 \text{ MN/m}\), respectively. Evidently, exterior ply 5 carried a larger load share. Without the other plies being monitored, the actual proportions cannot be assessed.

![Bridge 101 - Load 1@B - Individual Ply Deflection](image)

Fig. 3 Deflection measured relative to the ground for load 1 @ B

In span 3, the two east (west) plies 2 and 4 (plies 6 and 8) displaced an average of 4.19 mm (5.08 mm) and also showed repeatability. Plies 2 and 4 (plies 6 and 8) are two span continuous (simply supported) members. To approximate the different stiffnesses, a correction factor, CF, was used. CF is the ratio of the midspan displacement of a simply supported beam under a single midspan concentrated load to that of a two span beam under the same loading, i.e. \(CF = 1.4\). For purposes of comparing relative resistance, measured displacements for any two-span continuous plies were multiplied by 1.4. Adjusting for CF and MOE, the values for the east plies are \(1.4(0.0036 \text{ m} \times 14,400 \text{ MPa}) = 78.6 \text{ MN/m}\) and \(1.4(0.00478 \text{ m} \times 12,300 \text{ MPa}) = 58.6 \text{ MN/m}\). Within that chord, the exterior ply 4 evidently carried more load share. The values for the simply supported west plies are \(0.00582 \times 14,400 \text{ MPa} = 83.8 \text{ MN/m}\) and \(0.00434 \text{ m} \times 13,200 \text{ MPa} = 57.2 \text{ MN/m}\). Within that chord, the interior ply 6 evidently carried more load share. Overall, with the exception of ply 8, the load sharing appears similar. However,
support settlements (discussed in the report (1)) were included in the measurements and are an additional factor not discussed herein.

To assess load proportioning more fully, the load position (not shown) wherein the front axles of the locomotive were centered at midspan of Span 2 was used. The second axle was within Span 1 but near the interior support. All eight plies in Span 2 were monitored. Measurements were with respect to the displaced chord of the member (i.e. relative to the ends of the ply). Using the scaling for CF and MOE, the resulting adjusted displacements were then proportioned with respect to the summed values. However, in this case all plies were two-span continuous, so CF was not a distinguishing factor. The resulting portions for the east plies were 23%, 26%, 23% and 30%, from outer ply to inner ply, of the chord share. The values for the west plies were 18%, 17%, 31% and 34%, from inner to outer ply, of the chord share.

Similarly, when the upper limit (347 KN) of the ramp load was applied at midspan of Span 3, the resulting calculated proportions for the east plies were 25%, 28%, 24% and 24%, from outer ply to inner ply, of the chord share. The proportions for the west plies were 17%, 31%, 26%, and 25%, from outer ply to inner ply, of the chord share. In this case, some plies were simply supported and others were two-span continuous. When the load was applied at midspan of Span 1, the east (west) plies in that span resisted 34%, 24%, 21% and 21% (35%, 25%, 21%, and 20%), from inner ply to outer ply, of the chord share. These results are also based on measurements taken relative to the ground.

Lastly, the case of the 347 KN actuator load at midspan of Span 2 with measurements made relative to the displaced chord was examined. The east (west) plies in that span resisted 31%, 27%, 23% and 19% (20%, 19%, 29%, and 32%), from inner ply to outer ply, of the chord share.

7.2 Rolling train tests

With the electronic trigger data, the velocity of the train as well as its position at any time could be determined either directly or by interpolation. Figure 4 shows the displacement response for the entire bridge for the 4 car train in a position similar to that of the TLV train in Figure 3, however it was moving at 3.2 Km/hr. The ply displacements (relative to the ground) exhibit variation but are consistent with the loading. Figure 5 compares the responses (the eight ply values are averaged) for that same load position for both 3.2 Km/hr and 32 Km/hr velocities. On that basis, the outcomes are indistinguishable. However, Fig. 6 shows the same comparison of averaged values for the train being located in Span 2 and modest differences are evident at some data locations. While the rigorous determination of impact effects is complex, these plots suggest only a minor influence at these velocities.

Figures 7 and 8 show the transient responses at midspan of the two outer plies in an end span for both 3.2 Km/hr and 32 Km/hr velocities. While tightened slightly, due to different velocities, the peaks at 32 Km/hr are only modestly higher than those at 3.2 Km/hr. The 32 Km/hr peak responses are roughly 10% higher than the corresponding peaks at 3.2 Km/hr. Typically, for all the data locations, the
range was between 0% and 15%. Again, this is a general observation, not rigorous definition and computation.

Fig. 4 Deflection measured relative to ground for train speed of 3.2 Km/hr

Fig. 5 Comparison of deflections for 3.2 Km/hr Vs. 32 Km/hr – load position 3
Average Girder Displacements - Load Position 2
3.2 Km/hr and 32 Km/hr

![Graph showing vertical deflection relative to ground for 3.2 Km/hr and 32 Km/hr loads at position 2.]

Position measured from south end of bridge, (cm)

Fig. 6 Comparison of deflections for 3.2 Km/hr Vs 32 Km/hr – load position 2

Bridge Section AD - Position 3
Northerly Train Direction - Velocity = 3.2 Km/hr
Stringers 1,10 - Continuous Span

![Graph showing vertical displacements of stringers 1 and 10 for 3.2 Km/hr train velocity.]

Time, (seconds)

Fig. 7 Deflection response at location 3 for 3.2 Km/hr train velocity
Bridge Section AD - Position 3  
Northerly Train Direction - Velocity = 32 Km/hr  
Stringers 1,10 - Continuous Span

Fig. 8 Deflection response at location 3 for 32 Km/hr train velocity

7.3 Comparison of results of both tests

For each span, a comparison was made of the measured midspan displacement (relative to the ground) for the front axles of the locomotive centered at that midspan for 1) the TLV train statically placed, 2) the 4 car train moving at 3.2 Km/hr, and 3) the 4 car train moving at 32 Km/hr. The values for the TLV train test were scaled by 4/5 to account for the added stringers in the 4 car test. They were also adjusted proportionately to reflect the different train loads (for those axles actually on the span). Differences in axle spacing were neglected. The values for Span 1; (Span 2); (Span 3) were 4.17 mm, 4.45 mm, 4.80 mm; (not measured, 3.84 mm, 4.29 mm); (3.38 mm, 3.73 mm, 3.58 mm). Each is a mean value for all the plies monitored in the particular span.

8 Analytical modeling

The bridge is neither a sequence of simply supported spans nor a multi-span, fully continuous bridge. The staggered plies and, if present, midspan interconnecting ties cause a point load anywhere to have consequences to all other plies along the bridge. Even without support motions and degradation of the connections or end bearing, the load path is complex. Comprehensive work is in progress at several levels including 1) bounding an individual span response between fully fixed ends
and fully pinned ends, 2) modeling the grid structure including discontinuous ply ends, staggered placement and midspan links and c) full three dimensional modeling to include the rails and ties in the space frame. A semi-continuous beam model which, in essence, condenses the grid model to fewer members by assembling similar plies within a chord into aggregate members has been explored [4]. As an example of initial results, for the TLV actuator load (78 kips) at midspan of Span 2, the measured deflection corresponded to \( \Delta = PLVnEI \), where \( n = 75 \). Single span simple, two-span continuous, three-span continuous and single span fixed beam models imply \( n = 48, 67, 87 \) and \( 192 \), respectively. The semi-continuous beam model produced \( n = 71 \). This modeling and finding was for displacements measured relative to the displaced chord.

In some cases, the presence of support motions was significant, usually about 1.27 mm-2.54 mm. Figure 4 compares the idealized models of a simple span beam for the ends spans and a three span continuous beam to the data measured relative to the ground for Bridge 101. It is evident that support motion of that magnitude occurred and must be taken in to account to model absolute displacements. It appears the bridge was subjected to "a pumping like" displacement response, whereby the entire short length bridge structure was depressed "as a whole" by the heavy train loading.

9 Conclusions

Selected conclusions from the comprehensive report (1) are noted. Few cases of upward displacement were observed and support motion was a contributor. At some pile bents support motion was evident in the range of 1.2-2.54 mm downward. Empirical calculation reflecting MOE and span type showed a ply takes between 17% and 35% of chord loading. This changed moderately if support motion was removed. There was no pattern of load sharing among plies of a chord. After initial take up, response was linear from 134-347 KN. The sleeper beams stiffened Bridge 101 by an amount consistent with expectations. At train speeds up to 32 Km/hr, the impact effect was less than 15%.

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