DYNAMIC RESPONSE OF RAILROAD TIMBER BRIDGES

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ABSTRACT

Timber bridges are still commonly used by North American railroads. For short spans, they offer attractive alternatives to other types of bridges, because they are more economical, faster to construct and easier to maintain.

The current design practices do not allow an independent consideration of the effect of dynamic loads in sizing the bridge components.

The main objective of this paper is to describe the experimental work conducted to study the behaviour of timber bridge spans under the passage of trains at different speeds.

Based on the test results, the overall behaviour of two types of bridges, and the influence of different parameters on the dynamic factors are presented.

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INTRODUCTION

In the seventies, it was reported (Williams and Norton, 1976) that there were approximately 2,300 track miles of timber railroad bridges in service in the United States and Canada. Although their number has dropped since then due to replacements in other materials and line abandonments, they still represent a significant portion of the railroad bridge inventory.

For short spans, they offer an attractive alternative to other types of bridges, because they are more economical, faster to construct, and easier to maintain.

The current design practices (A.R.E.A. Manual, 1962) do not allow an independent consideration of the effects of the dynamic loads in sizing the bridge components, because there is very little information available on the dynamic behaviour of such structures. The only published literature found on the subject were reports by the Engineering Division of the Association of American Railroads in 1949 which dealt with exploratory tests on timber approaches as a part of dynamic tests conducted on steel bridges.

To study the dynamic response of timber railroad bridges, field tests were carried out to study the behaviour of two types of timber bridges (including the adjacent approaches and track sections) under the passage of trains at different speeds.

SELECTION OF TEST SITES

Two test sites were selected, one with a ballast deck bridge and another with an open deck bridge. The two sites were close to each other and were accessible by road and possessed single-storey height for ease of instrumentation. The sites chosen were approximately 25 miles northwest of Winnipeg near Grosse Isle, at Mi:16.50 and Mi:19.50 of Canadian National Railway's (CN) branchline named Oak Point Subdivision.

For each site, the behaviour of the bridge, the approach and the track were instrumented to measure the response.

Bridges

The first bridge is a slough crossing located at Mi:16.50 Oak Point Subdivision, consisting of a four-span ballast deck pile trestle with an overall length of 45'-10" and a height of 9'-4". It was built in 1943 using treated Douglas Fir material. The deck was made up of
10"x4" by 13'-6" long planks nailed onto ten 8"x16" spaced stringers (including two jack stringers) possessing an average span length of 11'-2½". The bents consist of a 12"x14" by 14'-0" long caps resting on five piles each driven with penetration varying from 18' - 24'.

A typical cross-section of the ballast deck bridge is shown in Figure 1(a).

The second bridge is a slough crossing at Mi:19.50 Oak Point subdivision, consisting of a three-span, open-deck pile trestle with an overall length of 36'-5½" and a height of 5'-4". It was built in 1945/46 using treated Douglas Fir material. Its deck is made up of twenty eight 8"x8" by 12'-0" bridge ties spaced at 12" centers which were renewed in 1975. They are resting on eight 8"x16" chorded stringers possessing an average span length of 11'-6½". The bents consist of a 12"x14" by 14'-0" long cap supported over five piles each, driven with a penetration of approximately 23'.

A typical cross-section of the open deck bridge is shown in Fig. 1(b).

Prior to testing, seemingly loose members were shimmed and all fasteners were tightened to ensure adequate performance of all components.

Approaches

The sections of track immediately behind the dumpwalls (within 15 feet) are referred to as "approaches". The approach sections of both bridges were in reasonable condition and possessed a full section of gravel and pit-run material. The approach of the ballast deck bridge had transition track ties.

Track

The sections of track beyond the approaches are referred to as the normal track sections. The alignments of track at both test sites were tangent. However, the grade at the first bridge was level, while the grade at the second was +0.02% North. The track consisted of 85 lb (Sec. 137 Algoma Canada MRS 85 lb HF-1944) jointed rails in lengths of 36 to 39 feet and 7½"x11" double shoulder tie plates spiked 8"x6" by 8'-0" ties spaced at approximately 22" centers and rested in ballast section of gravel and pit run material.

The zone speed over the stretch of track covered by the proposed tests was 30 mph with a maximum weight limit of 220,000 lbs for a 4-axle car. Therefore, to accommodate speeds up to 50 mph for the tests, the track had to be upgraded. This was done by spot surfacing and lining of track.
Figure 1(a). First test bridge - Ballast deck

Figure 1(b). Second test bridge - Open deck
TEST TRAINS

The test trains used for the tests were similar to the trains normally operated on this line for hauling limestone from Steep Rock, Manitoba. Since they were ordered at two different times, they differed in car numbers and car weights. Both were made up of a GR-20 series 4-axle diesel locomotive, two ballast-loaded open-top hopper cars and a caboose (Figure 2). The hopper cars possessed a transverse beam below the car body at mid-length which facilitated jacking for static tests. The test trains were scale weighed by their trucks at the local tower scale in CN's Symington Yard before leaving for the test sites. Table 1 gives the scale weights of locomotives and cars in the test trains.

Table 1. Scale Weights of Locomotives and Cars

<table>
<thead>
<tr>
<th>Description</th>
<th>Axial Weights (lbs)</th>
<th>Total Weights (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Leading</td>
<td>Trailing</td>
</tr>
<tr>
<td>Test Train #1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11 July, 1986</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Locomotive CN</td>
<td>124,220</td>
<td>123,560</td>
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<tr>
<td>#5516</td>
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<td></td>
</tr>
<tr>
<td>2. Hopper Car CN</td>
<td>101,740</td>
<td>104,700</td>
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<tr>
<td>#090151</td>
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<td></td>
</tr>
<tr>
<td>3. Hopper Car CN</td>
<td>96,090</td>
<td>101,700</td>
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<tr>
<td>#302360</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4. Caboose CN</td>
<td>31,300</td>
<td>31,520</td>
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<tr>
<td>#79384</td>
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<tr>
<td>Test Train #2</td>
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<td></td>
</tr>
<tr>
<td>16 Sept., 1986</td>
<td></td>
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<tr>
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INSTRUMENTATION

Both bridges, their approaches and normal track sections were instrumented to measure the loads at wheel-rail interfaces and the vertical displacements caused under rail points. The accelerations were also recorded at mid-points of bridge spans. Figures 3 shows the typical locations of the shear-load circuits used to measure the load at the wheel-rail interfaces, accelerometers, and the LVDT's for the vertical displacements of the ballast deck bridge.

Loads at Wheel-Rail Interfaces

The method (ORE Colloquia, 1969; Ahlbeck et al., 1980) used for measuring the vertical loads at the wheel-rail interfaces was based on a circuit consisting of eight strain gauges attached to the rail at each measurement location. Four gauges were installed on each side of the rail neutral axis as shown in Figure 4. The above pattern, often called a shear-load circuit, measures the net shear differential between the two gauged regions, a-b and c-d, with gauge pattern placed between the rail support points. The circuit output is directly proportional to the vertical load, P, as it passes between the gauges.

This strain gauge arrangement was tested in the Structural Laboratory of the University of Manitoba prior to its installation in the field and was found to exhibit excellent linearity and minimal
Figure 3. Location of instrumentation for first test bridge.

Figure 4. Arrangement of gauges in a typical shear-load circuit.
sensitivity to the lateral load (cross talk) or to the lateral position of the vertical load.

A total of six shear circuits were installed at each of the test sites: two circuits at the midspan of the bridge, two at the approach and two at the normal track at a distance of approximately 50 ft from the bridge.

Vertical Displacements

The vertical displacements were measured using LVDT's located at the same points at which where the shear-load circuits were installed. The LVDT's were installed under the rails and the chords of the stringers.

A 4" diameter PVC pipe was inserted into each augered hold at 8'-6" from the centerline of the track beneath each of the measurement points. In the center of the PVC pipe, a 2" diameter steel pipe was driven and the annular space between the pipes was kept hollow except at the top which was filled with a poly-foam ring and covered with plastic wrapping. This support system was used to minimize the transmission of vibrations generated in ground due to trains. The detail of the support system is shown in Figure 5. Four supports were installed at site #1 and three at site #2. A typical support system used for the second bridge is shown in Figure 6.

Acceleration

The accelerations were measured using two Bruel and Kjaer 4366 type accelerometers mounted to the underside of the stringer chords with Thermogrip hot-melt glue. The two accelerometers were connected to a pair of Bruel and Kjaer 2626 Conditioning Amplifiers which in turn were also hooked to the Data Acquisition System.

Data Acquisition System

A 16 Channel Techmar Lab Master D.A.S. connected to an IBM-PC Coprocessor was employed for recording loads, displacements and accelerations as measured from moving test trains. The rate of acquisition was 1600 readings per second per channel.

A Nicolet Explorer Digital Oscilloscope with 2 channels was used for selective viewing plots and storing information on loads at wheel-rail interfaces and vertical displacements during the tests.

A Hewlett-Packard Spectrum Analyzer equipped with X-Y Plotter was connected to the main circuitry for viewing and plotting the accelerations during the tests.
Figure 5. Support system for LVDT's.

Figure 6. LVDT support system used for second bridge.
An additional IBM-PC complete with printer and plotter was also available at the site to obtain hard copies of the data and various plots immediately after each test run.

The above arrangement allowed simultaneous measurements on 16 channels plus instant viewing of data on 4 channels.

The Data Acquisition System and other equipment were housed in a 40' long air-conditioned truck-trailer unit which had its own 5 KWH regulated power supply. A view of the truck and the equipment inside the trailer is as shown in Figure 7.

Figure 7. Test equipment in the truck-trailer.
TESTS

Field tests were carried out on two different days. On July 11, 1986, test series consisted of static and dynamic tests at site #1. The dynamic tests included runs of full test train followed by runs of locomotive at different speeds. On September 16, 1986, site #2 was tested in addition to repetition of some dynamic tests at site #1.

Calibration Tests

To determine the stiffness of the bridge, approach and track structures, static tests were conducted to calibrate the shear load circuits installed on the rails.

The middle of one CN car was centered over the load measurement locations. A load cell, a jack and a segmented railway car wheel were installed between the transverse beam of the car and the rail at each of the two rail points, as shown in Figures 8 and 9. The segmental wheels were used over rails to simulate the actual wheel-rail load conditions for static situations. The system was used to calibrate all the shear circuits installed at the different locations. The load was applied by means of a hand pump to a maximum of 30 kips.

Test Procedure

Tests on site #1 were conducted while the deck and the bridge timbers were wet. There was an unexpected amount of water under the bridges. The wet conditions also resulted in malfunction of a few gauges.

The dynamic tests were carried out with the test train #1 runs at crawl speed (1 mph), 5, 10, 15, 20, 30, 40, and 50 mph and measurements of loads, deflections, and accelerations were recorded and stored on floppy diskettes. The locomotive was uncoupled from the rest of the test train and tests were carried out with locomotive runs along at crawl speed, 5, 10, 20, 30, 40, and 50 mph, and the measurements were recorded and stored on floppy diskettes.

Due to the weather conditions at site #2 being worse than those at site #1, it was decided to postpone the remaining tests to another day.

The second series of tests took place on September 16, 1986. The tests commenced at site #2 after the gauges were installed and verified the day before. After calibrations of the load circuits, dynamic tests were carried out using test train #2 running at crawl speed, 5, 10, 15, 20, 30, 40, and 50 mph. Runs at crawl speed, 30 mph and 50 mph were repeated several times to record the data on different data acquisition systems.
Figure 8. Set-up for calibration test.

Figure 9. Calibration test in progress.
No uncoupling of the locomotive was attempted for the second site. The same test train was moved to test site #1. The dynamic tests were repeated at site #1 with test train #2 runs at crawl speed, 10, 30, and 50 mph. Similarly, a couple of additional runs were made at 30 and 50 mph to record data on different data acquisition systems.

For all the dynamic tests, the speed of the test trains was maintained by the engineman in the cabin. A Decatur Ray Gun Speed Measuring Device was also used to verify the actual test speeds.

TEST RESULTS

The experimental work at both sites involved twelve calibration tests and forty dynamic tests. Due to the massive amount of collected data, only a portion of the data and the highlights of the test results will be presented in this paper.

Calibration Tests

The calibration plots of the shear-load circuit at the mid-span of the bridge, the approach and the track for both sites are given in Figure 10. As it was expected, the bridge spans were stiffer than approaches and in turn, the approaches were stiffer than track sections. Similarly, by comparison, it can be noted that the ballast deck bridge was stiffer than the open deck bridge.

The test results indicated also that the load-displacement curves for the bridge spans were fairly linear, whereas those for the approaches and the track sections were nonlinear.

Loads at Wheel-Rail Interfaces

It should be noted that the loads at the wheel-rail contact interfaces may be influenced by the following:

1. static weight of the vehicle;
2. dynamic forces due to wheel-rail irregularities on the running surface, such as wheel out-of-roundness, wheel flats, and rail joints;
3. dynamic forces generated due to suspension system of the vehicle in motion such as roll, pitch and yaw;
4. track geometry irregularities, such as gauge, cross levels, surface and line; and
5. external disturbances such as wind, self-excited car hunting forces, and traction and braking forces.
Figure 10. Result of calibration test
Because of the foregoing, when a train is in motion, the loads at wheel-rail interfaces change continuously and thus they demonstrate wide variations. Figure 11 shows a typical output of the wheel loads versus time at mid-span of the second bridge at 30 mph.

The ratios of the measured wheel-rail contact loads to the static weights of wheels, i.e., dynamic load factor, \( \frac{L_d}{L_s} \), were calculated and plotted against the speed for the bridge spans, approaches and track sections. A typical behaviour at the mid-span of the open deck bridge and test train #2 is shown in Figure 12. It may be noted that the value of the dynamic load factors increases as the speed increases. The upper limit indicated a variation of 15% to 45% over a speed range of up to 50 mph.

These dynamic load factors were also calculated based on the various wheel loads for the above case, as shown in Figure 13. In general, these factors decrease with increase in the static wheel loads, which could be attributed to the fact that heavier axles are more stable and thus the weights of their wheels are evenly distributed, a condition which minimizes the vibrations due to the rolling action of vehicles.

**Vertical Displacements**

Figure 14 shows a typical output of the recorded vertical displacement at mid-span of the second bridge for test train #2 at 30 mph.

The ratios of the measured maximum displacement values to the computed static displacements, i.e., the dynamic displacement factors, \( \frac{\Delta d}{\Delta s} \), at mid-span points are related to the train speed as shown in Figure 15. The maximum static displacement was calculated assuming a simple span behaviour of the stringers. It may be noted that for the open deck bridge the value of these factors increases with an increase in speed, and varies from 2.1 to 2.6 over a speed of 50 mph. On the other hand, the speed up to 50 mph does not seem to have any effect for a ballast deck bridge for which an average value of 1.6 was obtained.

**Accelerations**

A typical output of the recorded acceleration versus time at the mid-span of the second bridge for test train #2 at 30 mph is shown in Figure 16. The behaviour indicated that the range of acceleration widened as the speed increased. For ballast deck bridge, the maximum acceleration ranged from +8.15 g to -6.78 g, but unfortunately for the open deck bridge at 20 mph and beyond, the range exceeded the measurement limits of the instrumentation which was set to +10.8 g.
Figure 11. Typical measured load versus time for the mid-span of second bridge at 30 mph.

Figure 12. Effect of speed on dynamic load factor.
Figure 13. Effect of static wheel load on dynamic load factor.

Figure 14. Typical vertical displacement with time at 30 mph - Second bridge.
Figure 15. Effect of speed on dynamic displacement factor.

Figure 16. Typical measured acceleration versus time.
SUMMARY

Based on the analysis of the data collected for the two bridges, the following conclusions could be drawn:

1. Factors such as track irregularities, wheel running surface irregularities, or rolling and hunting of cars in trains had a significant effect on loads at wheel-rail interfaces, vertical displacements, and accelerations.

2. The load-deflection behaviour of the bridges were found to be fairly linear, in contrast to the non-linear behaviour of the approaches and the track sections.

   The ballast-deck bridge was found to be comparably stiffer than the similar open deck one. Both bridges were substantially stiffer than the approaches, which, in turn, were substantially stiffer than the track sections.

3. For the open deck bridge, the dynamic load factor was found to increase with the increase of train speed. The dynamic load factor was also found to decrease with increased static wheel loads.

4. The measured displacements of the stringers were found to be significantly higher than the calculated displacements under static loading conditions. However, the stresses obtained from the measured displacement were still lower than the allowable values for the bridge type under consideration.

REFERENCES


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Figure 15. Effect of speed on dynamic displacement factor.