Response of timber bridges under train loading

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Timber bridges are still commonly used by several North American railroads. For short spans, they offer an attractive alternative to other types of bridges, as they are economical, faster to construct, and easy to maintain. Current design practices do not allow an independent consideration of the effects of the dynamic loads in sizing the bridge components, because very little information is available on the subject. Dynamic tests were carried out at two timber railroad bridge sites under the passage of trains at speeds varying from crawl, i.e., 1.6 km/h (1 mph), to 80.5 km/h (50 mph). The loads at wheel–rail interfaces, the vertical displacements, and the accelerations were measured at several locations on the bridge spans, the bridge approaches, and the normal track sections. The maximum values of the dynamic load factors obtained were 1.50, 1.65, and 1.85 for bridge, bridge approach, and normal track, respectively; and the corresponding maximum values of the dynamic displacement factors obtained were 1.30, 1.00, and 1.20. The main objective of this paper is to describe the experimental work and the influence on the measured values of the train speed and other factors.

Key words: railroad, timber, bridge, wheel–rail interfaces, load, deflection, frequency, load factor, dynamic displacement, track modulus.

Introduction

In the seventies, it was reported (Williams and Norton 1976) that there were approximately 3700 track kilometers (2300 track miles) of timber railroad bridges in service in the United States and Canada. Although their number has dropped since then owing to replacements with other materials and branch 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 economical, faster to construct, and easy to maintain.

The current design practices (American Railway Engineering Association 1988) do not allow an independent consideration of the effects of the dynamic loads in sizing the bridge components, because very little information is available on the subject. The only published literature on the subject was reported by the Engineering Division of the Association of American Railroads (1949a, b) on exploratory tests on timber railway bridge approaches as a part of extensive tests on steel bridges. A considerable variation was found in the magnitude of total impact. The percentage of impact (i.e., increase in stress over the static stress occurring at a slow speed), as determined from their tests for the stringer chord at the centre of the span, in one case had values as high as 57% at 74.4 km/h (46.2 mph) and in the other case, 35% at 56.3 km/h (35 mph).

To study the dynamic response, tests were carried out in 1986 on timber bridge spans at two test sites using test trains consisting of a locomotive unit, two loaded hopper cars, and a caboose. The loads at wheel–rail interfaces, the vertical displacements, and the accelerations were measured at several locations on the bridge spans, the bridge approaches, and the normal track sections at train speeds varying from crawl, i.e., 1.6 km/h (1 mph), to 80.5 km/h (50 mph).
Uppal et al. 1

(a) Slough Crossing at km 26.55 Oak Point Subdivision

- Base of Rail
- Ballast Deck

Track
Approach

0.26m
3.54m 3.44m 3.66m 3.02m 0.23m
14.15m

(b) Slough Crossing at km 31.38 Oak Point Subdivision

- Base of Rail
- Open Deck

Track
Approach

0.29m
3.54m 3.58m 3.51m 0.29m
11.13m

Fig. 1. (a) Ballast-deck bridge; (b) open-deck bridge.

Locomotive Hopper Car No. 1 Hopper Car No. 2 Caboose

AXIS NO.

AXIS SPACING

244 274 792 792 792 274 352 352 352 792 173 173 173 173 173
8'-0" 9'-0" 9'-0" 11'-4" 10'-8" 26'-0" 26'-0" 26'-0" 26'-0" 26'-0" 26'-0" 26'-0" 26'-0" 26'-0"

6'-0" 6'-0" 6'-0" 6'-0" 6'-0" 6'-0" 6'-0" 6'-0" 6'-0" 6'-0" 6'-0" 6'-0" 6'-0" 6'-0"

74 74 74 74 74 74 74 74 74 74 74 74 74 74

Fig. 2. Typical test train.
This paper gives a brief description of the test procedure, the test results, and the effect of the train speed and static wheel loads on the dynamic load and displacement factors.

Selection of test sites

Two test sites were selected, one having a ballast-deck bridge and the other having an open-deck bridge, which were close to each other, accessible by road, and possessed a single-story height for ease of instrumentation. The sites were approximately 40 km (25 miles) northwest of Winnipeg near Grosse Isle, Manitoba, at kilometers 26.55 and 31.38 (miles 16.50 and 19.50), respectively, of the Canadian National Railways' branch line, named the Oak Park Subdivision.

Bridges

The first bridge, located at kilometer 26.55 (mile 16.50), consisted of a four-span ballast-deck pile trestle with an overall length of 14.15 m (45 ft 10 in.) and a height of 2.85 m (9 ft 4 in.). It was built in 1943 using treated Douglas fir material. Its deck was made up of 25.4 cm x 10.2 cm x 4.12 m (10 in. x 4 in. x 13.5 ft) long transverse planks nailed onto ten 20.3 x 40.6 cm (8 x 16 in.) spaced stringers (including two jack stringers) possessing an average span length of 34.17 m (11 ft 2.3 in.). The majority of stringers were two spans long and alternately continuous over intermediate bents. Each bent consisted of a 30.5 cm x 35.6 cm x 4.27 m (12 in. x 14 in. x 14 ft) long cap resting over five timber round piles, having penetration varying from 4.88 to 7.12 m (16 to 24 ft).

The second bridge, located at kilometer 31.38 (mile 19.50), consisted of a three-span open-deck pile trestle with an overall length of 11.13 m (36 ft 5.5 in.) and a height of 1.63 m (5 ft 4 in.). It was built in 1945-1946 using treated Douglas fir material. Its deck was made up of thirty-six 20.3 cm x 11.13 cm x 3.66 m (8 in. x 8 in. x 12 ft) long bridge ties spaced at 30.5 cm (12 in.) centres, they were renewed in 1975. They rested on eight 20.3 cm x 40.6 cm (8 x 16 in.) chorded stringers having average span length of 3.52 m (11 ft 6.5 in.).

Bridge approaches and track sections

A section of track just past the bridge extremities which provides transition between the bridge and the normal track is referred to as a bridge approach. A section of track beyond a bridge approach (say 15.24 m (50 ft) or more) is referred to as the normal track. The track consisted of 378.1 N (85 lb) (Sec. 137 Algoma Canada MRS 85 lb. HF-1944) jointed rails in lengths of about 11.9 m (39 ft) and 19.0 x 28.0 cm (7.5 x 11 in.) double shoulder tie-plates spiked to 20.3 cm x 15.2 cm x 2.44 m (8 in. x 6 in. x 8 ft) long ties spaced at approximately 56.0 cm (22 in.) centres and embedded in a ballast section of gravel and pit-run material. The approach to the open-deck bridge had transition track ties. The condition of both the approaches and the track sections was good.

Test trains

The trains used for the tests were similar to the trains nor-
normally operated on this line for hauling limestone. They were made up of a GR20 series four-axle diesel locomotive, two ballast-loaded open-top hopper cars and a caboose as shown in Fig. 2. The open-top hopper cars possessed transverse beams at their mid-length just below their bodies, which facilitated jacking of the cars for static tests. The test trains were scale weighed by their trucks at the local tower scale in Canadian National Railways' Symington Yard before leaving for the test sites.

Table 1 gives the scale weights of the leading and the trailing trucks of the locomotives and cars in the test trains.

Instrumentation

The bridges, their approaches, and the normal track sections were instrumented to measure the loads at wheel—rail interfaces, the vertical displacements under the rail points, and the accelerations at the midpoints of the bridge spans, under the test trains moving at different speeds.

Figure 3 shows the typical location of the shear-load circuits used to measure the loads at the wheel—rail interfaces, the linear variable differential transducers (LVDTs) for the vertical displacements, and the accelerometers for accelerations at the open-deck bridge site.

Loads at wheel—rail interfaces

The method (ORE Colloquia 1969; Ahlbeck et al. 1980) employed for measuring the vertical loads at the wheel—rail interfaces was based on a shear circuit consisting of a gauge pattern attached to the rail support points (i.e., the gaps between the bridge ties or the track ties). The circuit measured the net shear differential between the two gauged regions, \( a-b \) and \( c-d \), which was directly proportional to the vertical load, \( P \), as it passed between the gauges as shown in Fig. 4.

A total of six shear circuits were installed at each of the test sites: two at the middle of one of the intermediate spans, two at the approach, and two at the normal track section as shown in Fig. 3b.

Vertical displacements

The vertical displacements were measured at the same points as where the loads at the wheel—rail interfaces were measured. The LVDTs were installed under the chords of stringers or under the rail bases. A special support system was used for the LVDTs to isolate their readings from the effects of the vibration of the ground. A typical example of the support system is given in Fig. 5. Four such supports were installed at the ballast-deck bridge site and three were installed at the open-deck site.

Accelerations

Vibrations due to accelerations were measured using two Bruel and Kjaer type 4366 accelerometers, which were mounted on the underside of the stringer chords. The accelerometers were connected to a pair of Bruel and Kjaer 2626 conditioning amplifiers which, in turn, were connected to the data acquisition system.

Data acquisition system

A 16-channel Techmar Lab Master data acquisition system (DAS) was employed for recording loads, displacements, and accelerations as measured from the moving test trains. The Lab Master was connected to an IBM-PC to convert analog data into digital data. The rate of acquisition was 600 readings per second per channel.

The DAS was supplemented by a Nicolet Explorer digital oscilloscope having dual channels, a Hewlett-Packard spectrum analyzer equipped with an X—Y plotter, and an additional IBM-PC with a printer and a plotter, which allowed the ability to view and plot the graphs as well as store digital information at the site immediately after each test.
The above equipment was housed in a 12.2 m (40 ft) long air-conditioned truck-trailer which had its own 5 kHz regulated power supply. A view of the truck-trailer unit and the inside of the trailer is given in Fig. 6.

Field tests

The tests in the field comprised both static and dynamic tests. The purpose of the static tests was to calibrate the system and to determine the relative stiffnesses of bridge spans, bridge approaches, and the normal track sections. The dynamic tests included runs of a full test train, followed by runs of the locomotive alone at different speeds. Their procedure is as below:

Calibration tests

The middle of one of the hopper cars was centred over one of the load measurement locations. A load cell, a jack, and a segmental railway car wheel were installed between the transverse beam of the car body and the rail at each of the two rail points as shown in Figs. 7 and 8. The segmented car wheel was used over the rails to simulate the actual wheel–rail contact conditions for static loading. The loads were applied gradually by means of hydraulic jacks operated by a hand pump to a maximum of 133.45 kN (30 kips) per rail and then lowered to zero. This procedure was used to calibrate all the shear load circuits.

Dynamic tests

The dynamic tests were carried at the ballast-deck bridge site with the test train No. 1 runs at crawl speed (i.e., 1.6 km/h (1 mph)), 8.0, 16.1, 24.1, 32.2, 48.3, 64.4, and 80.5 km/h (5, 10, 15, 20, 30, 40, and 50 mph). The measurements of loads, displacements, and accelerations were recorded and stored. Then the locomotive was uncoupled from the rest of the test train and tests were carried out with locomotive runs at crawl speed, 8.0, 16.1, 32.2, 48.3, 64.4, and 80.5 km/h; measurements were recorded and stored. Some of the gauges did not function properly owing to heavy rainfall, so the tests at the open-deck bridge were postponed to another time.

The second set of tests was carried out two months later. The tests were carried out at the open-deck bridge site. After the static tests, the dynamic tests were carried out with test train No. 2 running at crawl speed, 8.0, 16.1, 32.2, 48.3, 64.4, and 80.5 km/h. Separate tests using locomotive alone were not conducted at this site.

Following the completion of the test, the test train and truck-trailer were then moved to the site of ballast-deck bridge. The dynamic tests were repeated at the ballast-deck bridge site using test train No. 2 with runs at crawl speed, 16.1, 48.3, and 80.5 km/h.

For all dynamic tests, the speed of the test trains was maintained by the engineman in the cabin. A Decatur ray gun speed measuring device (i.e., a radar gun) was used to verify the actual test speeds. The readings from both sources corresponded well, except at speeds of 5 mph and less, for which the cabin speeds were considered to be more reliable.
Test results

The experimental work at both sites yielded a massive amount of data which were reviewed and analyzed. Only a sample of the data and the highlights of the findings are presented in this paper.

Calibration tests

The calibration plots of the shear-load circuits at the mid-span of the bridges, the approaches, and the track section for both sites are given in Fig. 9. It may be noted that the load-displacement curves for the bridge spans (S2R, S3R) were fairly linear, whereas those for the approaches (AR) and track (TR) sections were nonlinear, within the range of the measurements. Furthermore, the bridge spans were stiffer than the approaches, which were stiffer than the track sections. Magni-

![Fig. 9. Result of calibration test.](image)

<table>
<thead>
<tr>
<th>Location</th>
<th>Static deflection* (mm)</th>
<th>Stiffness (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Ballast-deck bridge site</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Span S3</td>
<td>2.60</td>
<td>54.29</td>
</tr>
<tr>
<td>Approach</td>
<td>9.72</td>
<td>14.52</td>
</tr>
<tr>
<td>Track section</td>
<td>11.86</td>
<td>11.90</td>
</tr>
<tr>
<td>(b) Open-deck bridge site</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Span S2</td>
<td>2.88</td>
<td>49.01</td>
</tr>
<tr>
<td>Approach</td>
<td>8.58</td>
<td>16.45</td>
</tr>
<tr>
<td>Track section</td>
<td>12.07</td>
<td>11.69</td>
</tr>
</tbody>
</table>

*Load = 141.14 kN.
Fig. 10. Typical measured load versus time for mid-span of open-deck bridge at 48.3 km/h (30 mph).

Table 3. Maximum measured loads, \( L_d \) (kN), at wheel-rail interfaces — test train No. 2

<table>
<thead>
<tr>
<th>Speed km/h (mph)</th>
<th>Span</th>
<th>Approach</th>
<th>Track</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Left rail</td>
<td>Right rail</td>
<td>Left rail</td>
</tr>
<tr>
<td>(a) Ballast-deck bridge site</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.6 (1)</td>
<td>153.77</td>
<td>152.31</td>
<td>151.82</td>
</tr>
<tr>
<td>16.1 (10)</td>
<td>158.13</td>
<td>—</td>
<td>152.26</td>
</tr>
<tr>
<td>48.3 (30)</td>
<td>168.31</td>
<td>160.14</td>
<td>180.73</td>
</tr>
<tr>
<td>80.5 (50)</td>
<td>158.80</td>
<td>160.14</td>
<td>205.51</td>
</tr>
<tr>
<td>(b) Open-deck bridge site</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.6 (1)</td>
<td>154.00</td>
<td>147.06</td>
<td>160.71</td>
</tr>
<tr>
<td>16.1 (10)</td>
<td>158.36</td>
<td>152.04</td>
<td>159.87</td>
</tr>
<tr>
<td>48.3 (30)</td>
<td>181.35</td>
<td>160.31</td>
<td>183.53</td>
</tr>
<tr>
<td>80.5 (50)</td>
<td>138.78</td>
<td>153.77</td>
<td>178.82</td>
</tr>
</tbody>
</table>

*Span S3 for ballast-deck bridge; span S2 for open-deck bridge.

Fig. 11. Typical vertical displacement versus time for mid-span of open-deck bridge at 48.3 km/h (30 mph).
TABLE 4. Maximum measured vertical displacements, $D_d$ (mm) — test train No. 2

| Speed km/h (mph) | Span S3 | | | Span S2 | | | Approach | | | Track |
|-----------------|--------|----------------|----------------|--------|--------|----------------|----------------|--------|----------------|
|                 | Left rail | Right rail | Left rail | Right rail | Left rail | Right rail | Left rail | Right rail |
| 1.6 (1)         | 5.22    | 4.03         | —         | 4.10     | —         | —         | —         | 11.92  | 10.14         |
| 16.1 (10)       | 5.35    | 4.11         | —         | 4.04     | —         | —         | —         | 12.27  | 9.89          |
| 48.3 (30)       | 5.46    | 4.00         | —         | 4.14     | —         | —         | —         | 12.43  | 9.83          |
| 80.5 (50)       | 5.39    | 4.17         | —         | 4.71     | —         | —         | —         | 13.31  | 11.12         |

(a) Ballast-deck bridge site

(b) Open-deck bridge site

1.6 (1) — — | 6.29 | 6.36 | 9.77 | 10.02 | 13.73 | 12.13 |
48.3 (30) — — | 7.54 | 6.43 | 9.45 | 10.16 | 13.87 | 13.12 |
80.5 (50) — — | 8.11 | 8.32 | 9.80 | 9.71 | 15.66 | 13.58 |

Fig. 12. Typical measured acceleration versus time for mid-span of open-deck bridge at 48.3 km/h (30 mph).
TABLE 5. Maximum and minimum accelerations (g) at midpoints of bridge spans* — test train No. 2

<table>
<thead>
<tr>
<th>Speed km/h (mph)</th>
<th>Left rail</th>
<th>Right rail</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maximum</td>
<td>Minimum</td>
</tr>
<tr>
<td>(a) Ballast-deck bridge site</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.6 (1)</td>
<td>+6.75</td>
<td>-1.06</td>
</tr>
<tr>
<td>16.1 (10)</td>
<td>+4.52</td>
<td>-5.18</td>
</tr>
<tr>
<td>48.5 (30)</td>
<td>+4.10</td>
<td>-4.86</td>
</tr>
<tr>
<td>(b) Open-deck bridge site</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.6 (1)</td>
<td>+0.23</td>
<td>-0.21</td>
</tr>
<tr>
<td>16.1 (10)</td>
<td>+5.78</td>
<td>-3.63</td>
</tr>
</tbody>
</table>

*Span S3 for ballast-deck bridge; S2 for open-deck bridge.

TABLE 6. Moduli of elasticity of bridge spans and moduli of track sections* (MPa)

<table>
<thead>
<tr>
<th>Span†</th>
<th>Approach</th>
<th>Track</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Ballast-deck bridge site</td>
<td></td>
<td></td>
</tr>
<tr>
<td>13169.0</td>
<td>6491.7</td>
<td>4978.0</td>
</tr>
<tr>
<td>(b) Open-deck bridge site</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10480.0</td>
<td>7345.04</td>
<td>862.9</td>
</tr>
</tbody>
</table>

*Modulus of elasticity of track is often referred to as track modulus.
†Span S3 for ballast-deck bridge; S2 for open-deck bridge.

TABLE 7. Upper limits of dynamic load factors, DLF — test train No. 2

<table>
<thead>
<tr>
<th>Speed km/h (mph)</th>
<th>Span*</th>
<th>Approach</th>
<th>Track</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Ballast-deck bridge site</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.6 (1)</td>
<td>1.25</td>
<td>1.23</td>
<td>1.13</td>
</tr>
<tr>
<td>16.1 (10)</td>
<td>1.28</td>
<td>1.17</td>
<td>1.23</td>
</tr>
<tr>
<td>48.3 (30)</td>
<td>1.28</td>
<td>1.47</td>
<td>1.40</td>
</tr>
<tr>
<td>80.5 (50)</td>
<td>1.49</td>
<td>1.61</td>
<td>1.86</td>
</tr>
<tr>
<td>(b) Open-deck bridge site</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.6 (1)</td>
<td>1.16</td>
<td>1.18</td>
<td>1.12</td>
</tr>
<tr>
<td>16.1 (10)</td>
<td>1.26</td>
<td>1.43</td>
<td>1.19</td>
</tr>
<tr>
<td>48.3 (30)</td>
<td>1.43</td>
<td>1.40</td>
<td>1.59</td>
</tr>
<tr>
<td>80.5 (50)</td>
<td>1.48</td>
<td>1.65</td>
<td>1.78</td>
</tr>
</tbody>
</table>

*Span S3 for ballast-deck bridge; S2 for open-deck bridge.

Fig. 13. Effect of train speed on dynamic load factor, DLF, at midpoint of open-deck bridge under test train No. 2. The measured load—deflection relationships were used to determine the moduli of elasticity of the bridge spans, the be probably due to the heavy damping caused by the layer of ballast. Contrary to this, in the case of the open-deck span S2, the displacements showed a definite increase with increase in the train speed.

Similarly, the values of the maximum measured displacements of the track sections indicated an increase with the increase of the train speed.

**Accelerations**

A typical output of the recorded acceleration versus time plot at the midpoint of the open-deck span S2 for the test train at 48.3 km/h (30 mph) is given in Fig. 12. The maximum and minimum values of the accelerations for the midpoints of the bridge spans at different speeds are given in Table 5. Values are given only up to train speed of 48.3 km/h (30 mph) for the ballast-deck bridge and 16.1 km/h (10 mph) for open-deck bridge owing to instrumentation problems.

**Analysis of test results**

**Calibration tests**

The maximum measured values of the vertical displacements for both sites for different speeds are given in Table 4. In the case of the ballast-deck span S3, the measured displacements showed no increase with increase in speed. This could sections. Figure 11 shows a typical measured vertical displacement versus time at mid-span of the open-deck bridge for test train No. 2 at 48.3 km/h (30 mph).

The maximum measured values of the vertical displacements for both sites for different speeds are given in Table 4. In the case of the ballast-deck span S3, the measured displacements showed no increase with increase in speed. This could
approaches, and the track sections. The calculated values based on these tests are given in Table 6. It may be noted that in the case of the open-deck bridge site, where special ties were used, the approach provided better transition between the track and the bridge span as opposed to the ballast-deck bridge site.

**Dynamic load factor (DLF = \( L_d / L_s \))**

The dynamic load factor, DLF, is the ratio of the maximum measured loads at wheel-rail interfaces, \( L_d \), to the corresponding static weights of their wheels, \( L_s \). The calculated values of DLF at different train speeds for the bridge span, the approaches, and the track sections are given in Table 7. A typical behaviour of the midpoint of the open-deck bridge span under test train No. 2 is shown in Fig. 13. It may be noted that the value of DLF increased as the train speed increased and that the upper limits indicated a variation of 16–48% over a speed range of 80.5 km/h (50 mph). Figure 14 shows the plot of the same value against the static wheel loads. It may be noted that the DLF decreased with increase in the static wheel loads. This meant that the magnitude of the impact was higher for lighter cars and lower for the heavier cars.

**TABLE 8. Dynamic load factors, DLF, for midpoints of bridge spans — test train No. 2**

<table>
<thead>
<tr>
<th>Speed km/h</th>
<th>Ballast-deck span S3</th>
<th>Open-deck span S2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Left rail</td>
<td>Right rail</td>
</tr>
<tr>
<td>1.6 (1</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>16.1 (10)</td>
<td>1.03</td>
<td>—</td>
</tr>
<tr>
<td>48.3 (30)</td>
<td>1.04</td>
<td>1.02</td>
</tr>
<tr>
<td>80.5 (50)</td>
<td>1.03</td>
<td>1.05</td>
</tr>
</tbody>
</table>

![Graph showing the effect of static wheel-load on dynamic load factor, DLF, at mid-span of open-deck bridge under test train No. 2.](image-url)
The static weights, $L_s$, of the wheels, were compared with the measured values, $L_{cr}$, at crawl speed, as given in Table 3. The ratios given in Table 8 indicate that $L_s$ and $L_{cr}$ are almost identical.

**Vertical displacement**

The measured vertical displacements of the bridge spans, given in Figs. 11 and 15, indicate that the vertical displacement consists of three parts, namely, (i) the rigid-body movement comprising play in the components resulting in tightening of the system and settlement of the support points under load; (ii) the static displacement caused by rolling loads; and (iii) the displacement attributed to the dynamic effect of the load.

For an open-deck span S3, at the train speed of 48.3 km/h (30 mph), the magnitude of the rigid-body movement is estimated as 1.96 mm as shown in Fig. 15. The rigid-body movements estimated for the two bridges are given in Table 9. It may be noted that there is more rigid-body movement in the components of the open-deck span as opposed to the ballast-deck span. The results indicated also that the movement depends on the train speed, with higher values at low speeds and lower values for high speeds.

The maximum measured net vertical displacements at midpoints of bridge spans were determined as the difference between the actual measured displacements and the displacements due to the rigid-body movements and are given in Table 10. Their values exhibit an increase with increase in the train speed.

**Dynamic displacement factor (DDF = $D_d/D_{cr}$)**

The dynamic displacement factor, DDF, is the ratios of the maximum measured net vertical displacements, $D_d$ (given in Table 10), to the maximum measured net vertical displacement at crawl speed, i.e., 1.6 km/h (1 mph), $D_{cr}$. The calculated values of DDF for the two bridges are given in Table 11. It may be noted that these factors increase with the increase in the train speed. The maximum values are 1.12 and 1.57 for the ballast-deck span and the open-deck span respectively.

**Damping of bridge span**

Since the behaviour of the bridge spans was linearly elastic and the fundamental mode dominated the free vibration, the logarithmic decrement technique was used on the free vibration portions of the acceleration versus time plots to compute the damping coefficients as a percentage of the critical damping. Their average values are given in Table 12. The damping coefficients did not show any definite relationship to the speed of the train. However, this could be attributed to the difficulty experienced in the acceleration measurements.

**Summary and conclusions**

Based on the measured data obtained from the field tests at the two different types of railway bridges, the following conclusions could be drawn:

1. The load—deflection behaviour of the bridges is fairly linear, in contrast to the nonlinear behaviour of the bridge approaches and the normal track sections.
2. The ballast-deck bridge span is comparably stiffer than the similar open-deck one. The bridge spans are stiffer than the approaches, which in turn were stiffer than the track sections.
3. For both types of bridge spans, the dynamic load factors, DLF, increased with increase in the train speed. Their maximum values were as follows: for ballast-deck bridge site, span S3 = 1.50, approach = 1.65, track = 1.85; and for open-deck bridge site, span S2 = 1.50, approach = 1.65, track = 1.80. These dynamic load factors decreased with increase in the static wheel loads.
4. For both spans, the dynamic displacement factors, DDF, increased with increase in the train speed. Their maximum values were found to be 1.12 and 1.57 for the ballast-deck span and the open-deck span respectively.
5. The range of acceleration widened with increase in train speed. Although both types of bridge spans appeared to be heavily damped, the damping in the ballast-deck span was found to be approximately 50% higher than that in the open-deck span.
TABLE 9. Estimated rigid-body movements (mm) at midpoints of bridge spans — test train No. 2

<table>
<thead>
<tr>
<th>Speed km/h (mph)</th>
<th>Ballast-deck span S3</th>
<th>Open-deck span S2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Left rail</td>
<td>Right rail</td>
</tr>
<tr>
<td>1.6 (1)</td>
<td>0.71</td>
<td>0.75</td>
</tr>
<tr>
<td>16.1 (10)</td>
<td>0.68</td>
<td>0.70</td>
</tr>
<tr>
<td>48.3 (30)</td>
<td>0.50</td>
<td>0.51</td>
</tr>
<tr>
<td>80.5 (50)</td>
<td>0.50</td>
<td>0.45</td>
</tr>
</tbody>
</table>

TABLE 10. Maximum measured net vertical displacements (mm) at midpoints of bridge spans — test train No. 2

<table>
<thead>
<tr>
<th>Speed km/h (mph)</th>
<th>Ballast-deck span S3</th>
<th>Open-deck span S2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Left rail</td>
<td>Right rail</td>
</tr>
<tr>
<td>1.6 (1)</td>
<td>4.52</td>
<td>3.32</td>
</tr>
<tr>
<td>16.1 (10)</td>
<td>4.55</td>
<td>3.41</td>
</tr>
<tr>
<td>48.3 (30)</td>
<td>4.80</td>
<td>3.45</td>
</tr>
<tr>
<td>80.5 (50)</td>
<td>4.89</td>
<td>3.72</td>
</tr>
</tbody>
</table>

TABLE 11. Dynamic displacement factors, DDF, for midpoints of bridge spans — test train No. 2

<table>
<thead>
<tr>
<th>Speed km/h (mph)</th>
<th>Ballast-deck span S3</th>
<th>Open-deck span S2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Left rail</td>
<td>Right rail</td>
</tr>
<tr>
<td>1.6 (1)</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
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</tr>
<tr>
<td>48.3 (30)</td>
<td>1.06</td>
<td>1.04</td>
</tr>
<tr>
<td>80.5 (50)</td>
<td>1.08</td>
<td>1.12</td>
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TABLE 12. Average damping coefficients of bridge spans

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Ballast-deck span S3</th>
<th>Open-deck span S2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Avg. damping coefficient (%)</td>
<td>9.8</td>
<td>6.2</td>
</tr>
<tr>
<td>Standard deviation</td>
<td>5.4</td>
<td>1.5</td>
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Acknowledgements

This study was carried out in the Department of Civil Engineering at the University of Manitoba with financial assistance from the Transportation Institute of the University of Manitoba. The authors are indebted to the Canadian National Railways Engineering Department for permission to use the test facilities. Special thanks to the Department of Civil Engineering technicians Messrs. E. Lemke and M. McVay for their assistance.


RESPONSE OF TIMBER BRIDGES UNDER TRAIN LOADING

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ABSTRACT

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

The current design practices do not allow an independent consideration of the effects 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.

Tests were conducted on two types of bridge spans, i.e. a ballast-deck and an open-deck. Based on the test results, the response of spans and the effects of other parameters such as speed and static wheel loads, etc., on dynamic factors are presented.
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 branch 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 easy to maintain.

The current design practices (A.R.E.A. Manual, 1986) 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 subject. 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 bridges under railway loading, field tests were carried out to measure the behaviour of two types of timber bridges (including the adjacent approaches and the track sections) under the passage of trains at different speeds. This paper presents
a brief description of the test procedure, the test results and the
effects of different parameters such as train speed and static wheel
loads on dynamic load and displacement factors.

SELECTION OF TEST SITES

Two test sites were selected, one with a ballast-deck bridge and
another with an open-deck bridge. Both sites were close to each
other, 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, Manitoba at Mi:16.50
and Mi:19.50 respectively, of Canadian National Railways' (CN)
branchline named Oak Point Subdivision.

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

Bridges
The first bridge was 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 transverse planks nailed onto ten 8"x16" spaced
stringers (including two jack stringers) possessing an average span
length of 11'-2-1/2". The majority of the stringers were two spans
long and alternatively continuous over intermediate bents. Each bent consists of a 12"x14" by 14'-0" long cap resting on five piles, driven to penetration varying from 18' - 24'.

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

The second bridge was 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'-5-1/2" and a height of 5'-4". It was built in 1945/46 using treated Douglas Fir material. Its deck was made up of twenty-eight 8"x8" by 12'-0" bridge ties spaced at 12" centres which were renewed in 1975. They were resting on eight 8"x16" chorded stringers possessing an average span length of 11'-6-1/4". The majority of the stringers were two spans long and alternatively continuous over intermediate bents. Each bent consisted of a 12"x14" by 14'-0" long cap supported over five piles each, driven to a penetration of approximately 23'.

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

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

A section of track behind the dumpwalls which provides transition between the track and the bridge (say within 15 feet of the dumpwalls), is referred to as an "approach". The approach sections of both bridges were in reasonable condition and possessed full sections of gravel and pit-run material. The approaches of the ballast-deck bridge had transition track ties.

Track Sections

A section of the track beyond the approaches (say about 50 feet from the dumpwalls and beyond) is referred to as the "normal track" section. The alignment of track at both test sites was tangent. 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 about 39 feet and 7-1/2"x11" double shoulder tie plates spiked to 8"x6" by 8'-0" long ties spaced at approximately 22" centres and embedded in ballast section of gravel and pit run material.

The zone speed over the stretch of track covered by these 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 was upgraded by spot surfacing and lining.
TEST TRAINS

The trains used for the tests were similar to the trains normally operated on this line for hauling limestone from Steep Rock, Manitoba. Since the trains were required at two different occasions, they differed in car numbers and car weights. But otherwise both of them were made up of a GR-20 series 4-axle diesel locomotive, two ballast-loaded open-top hopper cars and a caboose as shown in Figure 2. The hopper cars possessed transverse beams situated at their mid-length just below their bodies 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 prior to leaving for the test sites. Table 1 gives the scale weights of locomotives and cars in the test trains.

INSTRUMENTATION

The bridges, their approaches and the normal track sections were instrumented to measure the loads at wheel-rail interfaces and the vertical displacements under the rail points. The accelerations were also recorded at mid-points of the bridge spans. Figure 3 shows the typical locations of the shear-load circuits used to measure the load at the wheel-rail interfaces, and the LVDTs for the vertical displacements and the accelerometers for the first test site.
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 of the measurement locations. Four gauges were installed on each side of the rail neutral axis as shown in Figure 4. The above pattern, referred to as 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 it was found to exhibit excellent linearity and minimal sensitivity to the lateral load (cross talk) or to the lateral component of the vertical load.

A total of six shear circuits were installed at each of the test sites: two circuits at the middle of the intermediate span of the bridge, two at the approach and two at the normal track section at an approximate distance of 50 feet from the bridge.

Vertical Displacements

The vertical displacements were measured using LVDTs at the same points where the shear-load circuits were installed. The LVDTs were
mounted under the chords of the spans and under the rails for the approaches and normal the track sections. Four inch diameter PVC pipes were pushed into augered holes located at 8'-6" from the centreline of the track and beneath the measurement points. A two inch diameter steel pipe was inserted into each of the PVC pipes and driven into the ground. The annular spaces between the pipes were kept hollow except at the top where they were filled with poly-foam rings and covered with plastic wrappings. This type of support system was used to prevent any ground vibrations produced by train dynamics from affecting the LVDT readings. The detail of support systems is shown in Figure 5. Four such 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.

**Accelerations**

The accelerations were measured using two Bruel and Kjaer 4366 type accelerometers which were 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 Data Acquisition System connected to an IBM-PC Coprocessor was employed for recording loads, displacements and accelerations as measured from the moving test trains. The rate of acquisition was 1600 readings per second for one channel.
A Nicolet Explorer Digital Oscilloscope having two channels was used for selective viewing of plots and their 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. An additional IBM-Personal Computer complete with printer and plotter was also available at the site to obtain hard copies of the data and the time plots immediately after each test run.

The above arrangement permitted simultaneous recording of measurements on sixteen channels plus instant viewing and storing selective information on another four channels.

The data acquisition system and other pieces of equipment were housed in a 40 feet 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 shown in Figure 7.

TESTS

Field tests were carried out on two different days. On 11 July 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 16 September 1986, site #2 was
tested in addition to repetition of some of the dynamic tests at site #1.

Calibration Tests
Static tests were conducted to calibrate the shear-load circuits installed on the rails as well as to determine the load-displacement characteristics of the bridges, the approaches and the track sections.

The mid-point of one of the hopper cars was centered over one of the load measurement locations. A load cell, a jack and a segmented railway car wheel were placed between the transverse beam of the car body and the rail at each of the two rail points, as shown in Figures 8 and 9. The segmented wheels were used on rails to simulate the actual wheel-rail load conditions for static situations. This system was used to calibrate all the shear circuits installed at the different locations. The loads were applied by means of hydraulic jacks operated by a hand pump to a maximum of 30 kips per rail.

Test Procedure
Tests on site #1 were conducted while the deck and the bridge timbers were wet due to heavy rainfall. There was also an unexpected amount of water under the bridges. These conditions resulted in malfunction of a few gauges. The dynamic tests were carried out with the test train #1 runs at crawl speed (i.e.,1 mph), 5, 10, 15, 20, 30, 40 and 50 mph and measurements of loads, displacements and accelerations were recorded and stored on floppy diskettes. The locomotive was
then uncoupled from the rest of the test train and tests were carried out with locomotive runs alone at crawl speed, 5, 10, 20, 30, 40 and 50 mph, and the measurements were recorded and stored on diskettes.

Because the weather conditions at site #2 became worse than those existing at site #1, decision was made to postpone the remaining tests to another day.

The second series of tests took place on 16 September 1986. The tests commenced at site #2 after the gauges were installed and verified the day before. The calibrations of the load circuits were done first and then the dynamic tests were carried out using test train #2 runs 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 and some of the data was also recorded on Nicolet Oscilloscope for comparison with the one stored on Techmar Lab Master.

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. Again, a few additional runs were made at 30 and 50 mph and some of the data was also recorded on Nicolet Oscilloscope.

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 (i.e. a radar) was also used to verify the actual test speeds. Readings from both sources corresponded very well except at speeds of 5 mph and below for which the cabin readings were found to be more reliable.

**TEST RESULTS**

The experimental work at both sites involved twelve calibration tests and forty dynamic tests. These yielded a massive amount of data, the full treatment of which is beyond the scope of this paper. Therefore only a sample of the data and the highlights of some of the findings will be presented here.

**Calibration Tests**

The calibration plots of the shear-load circuit at the mid-span of the bridge, the approach and the track section for both sites are given in Figure 10. It was found that the bridge spans were stiffer than approaches and in turn, the approaches were stiffer than track sections. Similarly, the ballast-deck bridge span was found to be stiffer than the open-deck bridge span.

The test results also indicated that the load-displacement curves for the bridge spans were fairly linear, whereas those for the approaches and the track sections were non-linear, within the range of the measurements.
Loads at Wheel-Rail Interfaces

The loads at the wheel-rail contact points for a railway vehicle in motion may depend on the following factors:

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 bounce, sway, roll, pitch and yaw, etc.;

4. track geometry irregularities, such as gauge, cross levels, surface and line;

5. external disturbances such as wind, self-excited car hunting forces, and traction and braking forces; and

6. speed of the vehicle.

And when the vehicle passes over a bridge span, the characteristics of the span also affect the loads at wheel-rail interfaces which continuously fluctuate about their static values. Figure 11 shows a typical plot of the loads versus time for mid-span of second bridge at 30 mph. The influence of some of the above mentioned factors is evident from the variation of values of loads with respect to time at the two contact points.

Table 2 gives the maximum measured values of the loads at wheel-rail interfaces.
The ratios of the measured wheel-rail contact loads to the static weights of wheels, i.e., dynamic load factor, DLF = Ld/Ls, were calculated and plotted against the speed for the bridge spans, the approaches and the track sections. A typical behaviour at the mid-point of the open-deck bridge span for test train #2 is shown in Figure 12. It may be noted that the values of the dynamic load factors increased as the speed increased. The upper limit indicated a variation of 16% to 49% over a speed range of up to 50 mph.

These dynamic load factors were also plotted against the static wheel loads and for the above case, are shown in Figure 13. In general, the DLFs decreased with increase in the static wheel loads. This may be attributed to the fact that heavier axles are more stable as the weights of their wheels are more evenly distributed, a condition which helps reduce the vibrations due to the rolling action of vehicles.

**Vertical Displacements**

Figure 14 shows a typical plot of the measured vertical displacement versus time at mid-span of the second bridge for test train #2 at 30 mph. Table 3 gives the maximum measured values of the vertical displacements.

The ratios of the measured maximum displacement values to the
computed static displacements as well as the displacements at crawl speed, i.e., the dynamic displacement factors, $\text{DDF} = \frac{Dd}{Dsc}$ and $\text{DDF} = \frac{Dd}{Dcr}$ respectively, for mid-points of the spans were plotted against train speed and are shown in Figure 15.

The values of the maximum static displacements were calculated assuming behaviour of the chords as simply supported beams. It may be noted that for the open-deck bridge span the value of the DDFs increased with an increase in the speed, i.e., $\frac{Dd}{Dsc}$ varied from 2.1 to 2.7 and $\frac{Dd}{Dcr}$ from 1.0 to 1.3 over a speed of 50 mph. On the other hand, speeds of up to 50 mph did not seem to indicate any effect on the ballast-deck bridge span for which the average values of $\frac{Dd}{Dsc} = 1.7$ and $\frac{Dd}{Dcr} = 1.0$ were obtained.

**Accelerations**

A typical output of the measured acceleration versus time at the mid-span of the second bridge for test train #2 for 30 mph is shown in Figure 16. It was noted that the range of the measured accelerations widened as the speed increased. For ballast-deck bridge, the maximum acceleration ranged from $+10.08 \, \text{g}$ to $-7.00 \, \text{g}$, but unfortunately for the open deck-bridge at 20 mph and beyond, the range exceeded the measurement limits of the instrumentation which was set at $\pm10.08 \, \text{g}$.
Damping in Bridge Spans

The logarithmic decrement technique was employed to the free vibration portion of the acceleration versus time plots for midpoints of the bridge spans to compute the damping coefficients as a percentage of the critical damping. There was a fair amount of spread in the values obtained. However, the average values of the coefficients were found to be as follows:

Ballast-deck Span S3 = 9.8%
Open-deck Span S2 = 6.2%

SUMMARY

Based on the analysis of the data obtained from the tests at the two sites, the following conclusions were drawn:

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

2. The load-deflection behaviour of the bridge spans was found to be fairly linear, in contrast to the non-linear behaviour of the approaches and the track sections. The ballast-deck bridge span was found to be stiffer than the open-deck one. Both bridge spans were substantially stiffer than the approaches, which, in
turn, were stiffer than the track sections.

3. For both types of bridge spans, the dynamic load factors, DLF, were found to increase in value with the increase of train speed. The maximum value of DLF measured was 1.49 at 50 mph. The dynamic load factors were also found to decrease with increase in static wheel loads.

4. For the open deck span, the dynamic displacement factors, DDF, increased with increase in speed with a maximum value of 1.316 over the crawl speed. On the other hand, speeds of up to 50 mph did not show any effect on the ballast deck span.

5. The range of acceleration widened with increase in the train speed. At speeds beyond 20 mph, the values started to exceed the measurement range of ±10.08 g.

6. Although both types of bridge spans appeared to be heavily damped, damping in the ballast deck span was approximately 50% more than the damping in open deck span.

ACKNOWLEDGEMENTS

This study was carried out in the Department of Civil Engineering at the University of Manitoba with financial assistance from the Transportation Institute of the University of Manitoba. The authors are indebted to the Canadian National Railway and, in particular, to the Regional Chief Engineer, Mr. R. D. Miles for permission to use the test facilities.
REFERENCES


Table 1. Scale weights of locomotives and cars.

<table>
<thead>
<tr>
<th>Description</th>
<th>Truck 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 11 July 1986</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Locomotive CN #5516</td>
<td>124,220.</td>
<td>123,560.</td>
</tr>
<tr>
<td>3. Hopper car CN #302360</td>
<td>96,090.</td>
<td>101,700.</td>
</tr>
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<td>Test train #2 16 Sept. 1986</td>
<td></td>
<td></td>
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<tr>
<td>1. Locomotive CN #5608</td>
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<td>2. Hopper car CN #090159</td>
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</tr>
<tr>
<td>3. Hopper car CN #090151</td>
<td>100,840.</td>
<td>108,760.</td>
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Table 2. Maximum measured loads at wheel-rail interfaces.

a) Test site #1 - test train #2

<table>
<thead>
<tr>
<th>Speed (mph)</th>
<th>Span #S3</th>
<th>Approach Track</th>
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<td></td>
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<td>1</td>
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<td>50</td>
<td>31.73</td>
<td>36.00</td>
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b) Test site #2 - test train #2

<table>
<thead>
<tr>
<th>Speed (mph)</th>
<th>Span #S2</th>
<th>Approach Track</th>
</tr>
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<td>Dynamic Static</td>
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<td>30</td>
<td>31.45</td>
<td>40.17</td>
</tr>
<tr>
<td>50</td>
<td>31.73</td>
<td>34.57</td>
</tr>
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</table>

Table 3. - Maximum measured vertical displacements (mm).

a) Test site #1 - test train #2

<table>
<thead>
<tr>
<th>Speed (mph)</th>
<th>Span #3</th>
<th>Span #2</th>
<th>Track</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>L Rail</td>
<td>R Rail</td>
<td>L Rail</td>
</tr>
<tr>
<td>1</td>
<td>5.22</td>
<td>4.03</td>
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<tr>
<td>30</td>
<td>5.46</td>
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<td>-</td>
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<tr>
<td>50</td>
<td>5.39</td>
<td>4.17</td>
<td>-</td>
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b) Test site #2 - test train #2

<table>
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<th>Speed (mph)</th>
<th>Span #2</th>
<th>Approach</th>
<th>Track</th>
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<tbody>
<tr>
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Figure 1(a). First test bridge - Ballast deck.

Figure 1(b). Second test bridge - Open deck.
Figure 2. Typical test train.
Figure 3. Location of instrumentation for first test bridge.

Figure 4. Arrangement of gauges in a typical shear-load circuit.
Figure 5. Support system for LVDTs.

Figure 6. LVDT support system used for second bridge.
Figure 7. Test equipment in the truck-trailer.
Figure 8. Set-up for calibration test.

Figure 9. Calibration test in progress.
Figure 10. Result of calibration test.

Figure 11. Typical measured load versus time for 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 versus time for mid-span of second bridge at 30 mph.

Figure 15. Effect of speed on dynamic displacement factor.
Figure 16. Typical measured acceleration versus time for mid-span of second bridge at 30 mph.
Figure 6. Test equipment in the truck-trailer.
Figure 5. Typical support system for LVDT's
Figure 8. Calibration test in progress.
Figure 2. Typical test train.