# 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 attractive alternatives to other types of bridges because they are more economical, faster to construct, and easy to maintain. Current design practices do not allow independent consideration of the effects of dynamic loads in sizing bridge components. The main objective of this paper is to describe the experimental work conducted to study the behavior of timber bridge spans under the passage of trains at different speeds. Tests were conducted on two types of bridge spans, a ballast deck and an open deck. Test results indicate the response of spans and the effects of other parameters such as speed and static wheel loads to dynamic factors.**

In the 1970s it was reported (*1*) 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 as a result of replacement by 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.

Current design practices (*2*) do not allow independent consideration of the effects of dynamic loads in sizing bridge components, because there is little information available on the subject. The only published literature found was reports by the Engineering Division of the Association of American Railroads (*3, 4*) that 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 behavior of two types of timber bridges (including the adjacent approaches and the track sections) under the passage of trains at different speeds. This paper is 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. The two sites were close to each other, were accessible by road, and were of single-storey height for ease of instrumentation. The sites chosen were approximately 25 mi northwest of Winnipeg near Grosse Isle, Manitoba, at Mile 16.50 and Mile 19.50, respectively, of the Canadian National Railways (CN) branch line named Oak Point Subdivision. At each site, the bridge, the approach, and the track section were instrumented to measure the response.

### Bridges

The first bridge was a slough crossing, located at Mile 16.50 Oak Point Subdivision, that was a four-span ballast-deck pile trestle with an overall length of 45 ft 10 in. and a height of 9 ft 4 in. It was built in 1943 using treated Douglas Fir material. The deck was made up of 10 in. × 4 in. by 13 ft 6 in. long transverse planks nailed onto ten 8- × 16-in. spaced stringers (including two jack stringers) with an average span length of 11 ft $2^1/_2$ in. A majority of the stringers were two spans long and alternatively continuous over intermediate bents. Each bent consists of a 12 in. × 14 in. by 14 ft 0 in. long cap resting on five piles, driven to penetrations varying from 18 to 24 ft. A typical elevation and cross section of the ballast-deck bridge are shown in Figure 1 (top).

The second bridge was a slough crossing, at Mile 19.50 Oak Point Subdivision, consisting of a three-span, open-deck pile trestle with an overall length of 36 ft $5^1/_2$ in. and a height of 5 ft 4 in. It was built in 1945–1946 using treated Douglas Fir material. Its deck was made up of twenty-eight 8 in. × 8 in. by 12 ft 0 in. bridge ties spaced at 12-ft centers, which were renewed in 1975. They were resting on eight 8 in. × 16 in. chorded stringers with an average span length of 11 ft $6^1/_4$ in. A majority of the stringers were two spans long and alternatively continuous over intermediate bents. Each bent consisted of a 12 in. × 14 in. by 14 ft 0 in. long cap supported over five piles, each driven to a penetration of approximately 23 ft. A typical elevation and cross section of the open-deck bridge are shown in Figure 1 (bottom).

Before 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 ft of the dumpwalls), is referred to as an "approach." The approach sections of both bridges were in reasonable condition and

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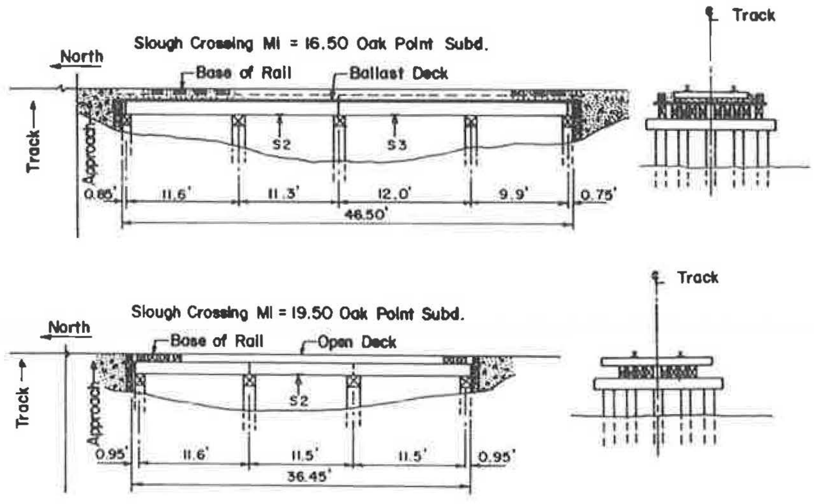


**FIGURE 1 First test bridge—ballast deck (top); second test bridge—open deck (bottom).**

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 ft from the dumpwalls and beyond) is referred to as a "normal track" section. The alignment of track at both test sites was tangent. The grade at the first bridge was level, and the grade at the second was +0.02 percent north. The track consisted of 85-lb (Sec. 137 Algoma Canada MRS 85-lb HF-1944) jointed rails in lengths of about 39 ft and $7^1/_2$ in. × 11 in. double shoulder tie plates spiked to 8 in. × 6 in. by 8 ft 0 in. long ties spaced at approximately 22-ft centers and embedded in a 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 lb for a four-axle car. Therefore, to accommodate speeds of 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. Because trains were required on two different occasions, they differed in car numbers and car weights. However, both of them were made up of a GR-20 series four-axle diesel locomotive, two ballast-loaded open-top hopper cars, and a caboose as shown in Figure 2. The hopper cars had transverse beams situated at their midlength 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 before they left 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. Accelerations were also recorded at midpoints of the bridge spans. Figure 3 shows typical locations of the shear-load circuits used to measure the load at the wheel-rail interfaces, the linear velocity displacement transformers (LVDTs) for the vertical displacements, and the accelerometers for the first test site.

### Loads at Wheel-Rail Interfaces

The method (*5, 6*) 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. This pattern, referred to as a shear-load circuit, measures the net shear differential between the two gauged regions, a-b and c-d, with a 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 before 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 ft from the bridge.

### Vertical Displacements

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 track sections. PVC pipes 4 in. in diameter were pushed into augered holes located 8 ft 6 in. from the centerline of the track and beneath the measurement points. A steel pipe 2 in. in diameter was inserted into each of the PVC pipes and driven into the ground. The annular spaces between the pipes were kept hollow except


**FIGURE 2 Typical test train.**

TABLE 1 SCALE WEIGHTS OF LOCOMOTIVES AND CARS

| Description | | Truck Weights (lbs) Leading | Truck Weights (lbs) Trailing | Total Weights (lbs) |
|---|---|---|---|---|
| Test train #1 11 July 1986 | 1. Locomotive CN #5516 | 124,220. | 123,560. | 247,700. |
| | 2. Hopper car CN #090151 | 101,740. | 104,700. | 206,440. |
| | 3. Hopper car CN #302360 | 96,090. | 101,700. | 197,760. |
| | 4. Caboose CN #79384 | 31,300. | 31,520. | 62,820. |
| Test train #2 16 Sept. 1986 | 1. Locomotive CN #5608 | 126,900. | 125,800. | 252,760. |
| | 2. Hopper car CN #090159 | 88,480. | 98,700. | 187,180. |
| | 3. Hopper car CN #090151 | 100,840. | 108,760. | 204,600. |
| | 4. Caboose CN #79715 | 30,580. | 30,240. | 60,820. |

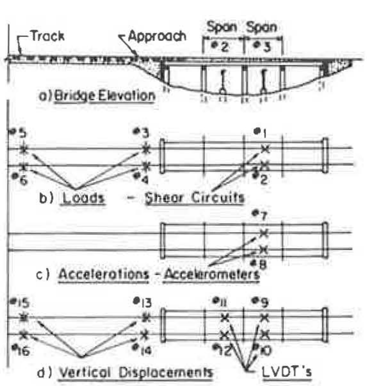


**FIGURE 3 Location of instrumentation for first test bridge.**

at the top where they were filled with polyfoam 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

Accelerations were measured using two Bruel and Kjaer 4366 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

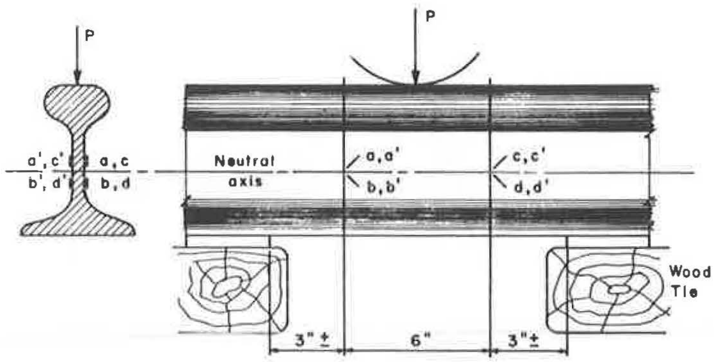


FIGURE 4 Arrangement of gauges in a typical shear-load circuit.

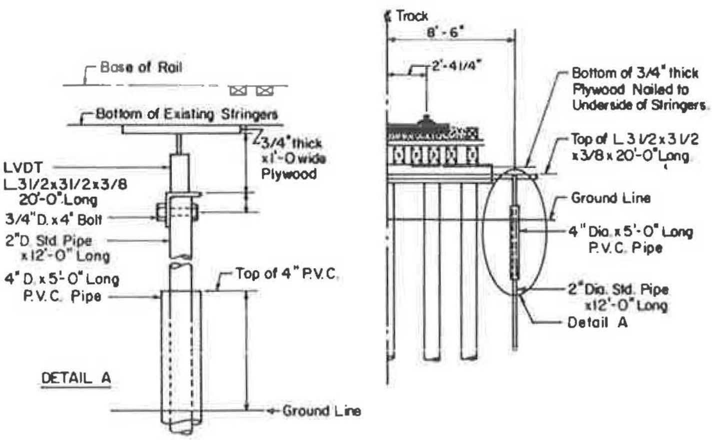


FIGURE 5 Support system for LVDTs.

amplifiers that, 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 measured from the moving test trains. The rate of acquisition was 1,600 readings per second for one channel. A Nicolet Explorer digital oscilloscope with two channels was used for selective viewing of plots and storing information on loads at wheel-rail interfaces and vertical displacements during the tests.

A Hewlett-Packard spectrum analyzer equipped with an 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 time plots immediately after each test run. This arrangement permitted simultaneous recording of measurements on 16 channels plus instant viewing and storing of selective information on another 4 channels.

The data acquisition system and other pieces of equipment were housed in an air-conditioned truck-trailer unit 40 ft long that had its own 5-kWh regulated power supply. A view of the truck and the equipment inside the trailer is shown in Figure 7.

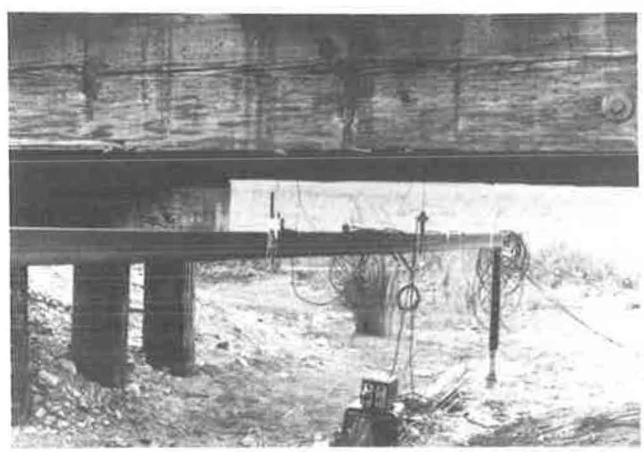

FIGURE 6 LVDT support system for second bridge.

## TESTS

Field tests were carried out on two different days. On July 11, 1986, series of static and dynamic tests were conducted at Site 1. The dynamic tests included runs of a full test train followed by runs of a locomotive at different speeds. On September 16, 1986, tests were run at Site 2, and some of the dynamic tests at Site 1 were repeated.

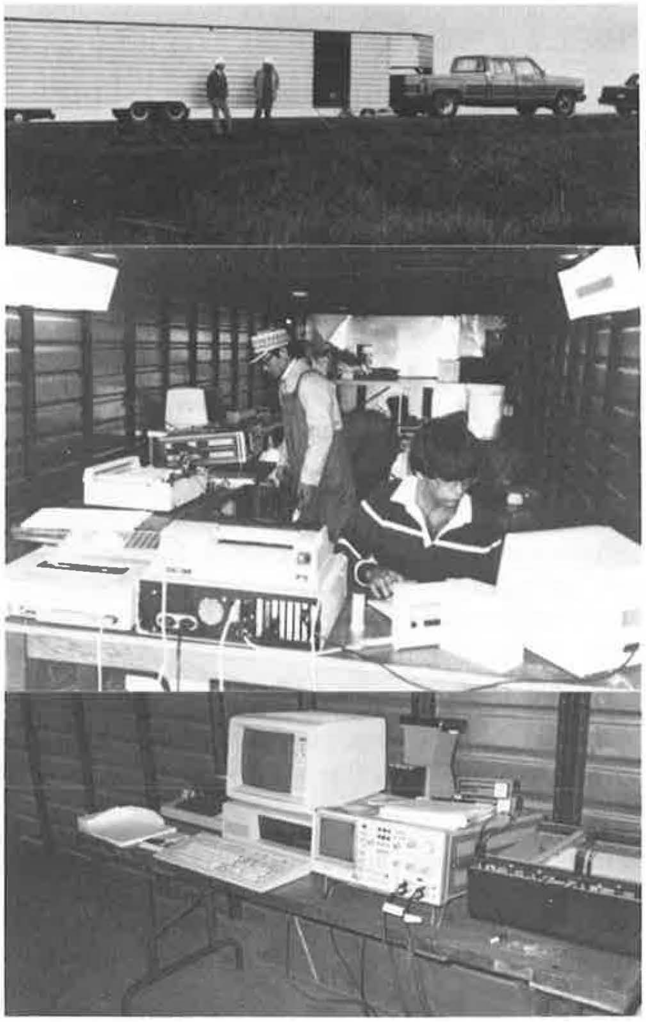

FIGURE 7 Test equipment in truck trailer.

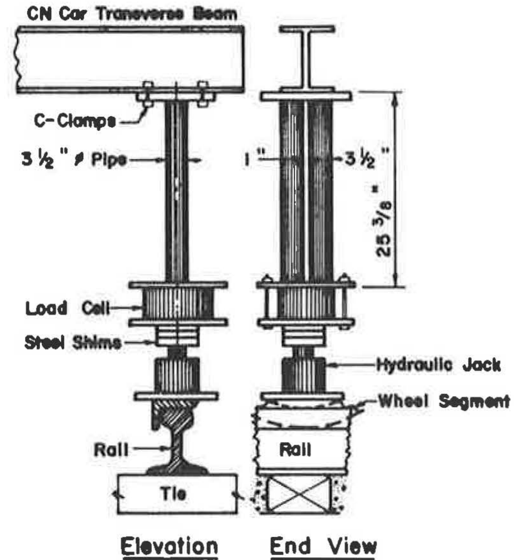


FIGURE 8 Setup for calibration test.

FIGURE 9 Calibration test in progress.

### 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 midpoint of one of the hopper cars was centered over one of the load measurement locations. A load well, a jack, and a segmented railway car wheel were placed between the transverse beam of the carbody 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 of the shear circuits installed at both locations. The loads were applied by hydraulic jacks operated by a hand pump to a maximum of 30 kips per rail.

### Test Procedure

Tests at Site 1 were conducted while the deck and the bridge timbers were wet after a 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 Test Train 1 running 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 the locomotive running alone at crawl speed, 5, 10, 20, 30, 40, and 50 mph, and the measurements were recorded and stored on diskettes.

Because weather conditions at Site 2 became worse than than they had been at Site 1, it was decided to postpone the remaining tests until another day.

The second series of tests took place on September 16, 1986. The tests commenced at Site 2 after the gauges had been installed and verified the day before. Calibration of the load circuits was done first, and then the 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 and 30 and 50 mph were repeated several times, and some of the data were also recorded on the Nicolet oscilloscope for comparison with those stored on the Techmar Lab Master. No uncoupling of the locomotive was attempted at 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 running 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 were also recorded on the Nicolet oscilloscope. 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) was also used to verify the actual test speeds. Readings from both sources corresponded 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 12 calibration tests and 40 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 midspan of the bridge, the approach, and the track section at both sites are shown in Figure 10. It was found that the bridge spans were stiffer than the approaches and, in turn, the approaches were stiffer than the 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 nonlinear, 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 by the suspension system of the vehicle in motion, such as bounce, sway, roll, pitch, and yaw;
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.

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 loads versus time for the midspan of the second bridge at 30 mph. The influence of some of the previously 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. Typical behavior at the midpoint of the open-deck bridge span under Test Train 2 is shown in Figure 12. It may be noted that the values of the dynamic load factors increase as the speed increases. The upper limit indicates a variation of from 16 to 49 percent for speeds of up to 50 mph.

These dynamic load factors (DLFs) were also plotted against the static wheel loads (Figure 13). In general, DLFs decrease with an increase in static wheel loads. This may be because heavier axles are more stable because the weights of their wheels are more evenly distributed, a condition that 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 midspan 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, DDF = Dd/Dsc and DDF = Dd/Dcr, respectively) for mid-

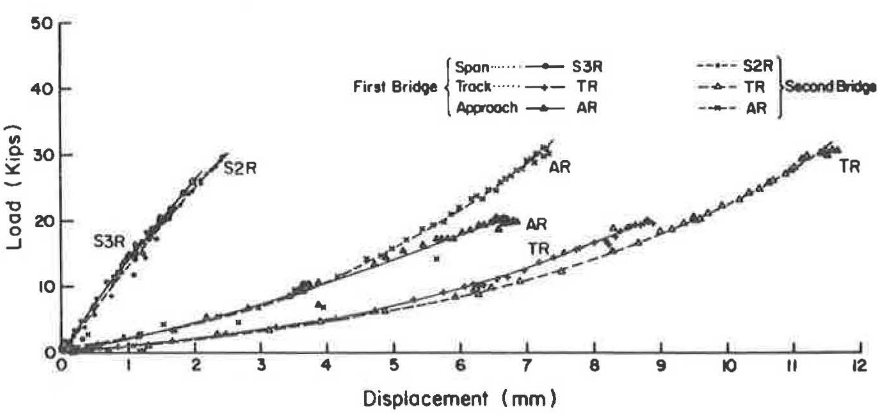


FIGURE 10 Results of calibration test.

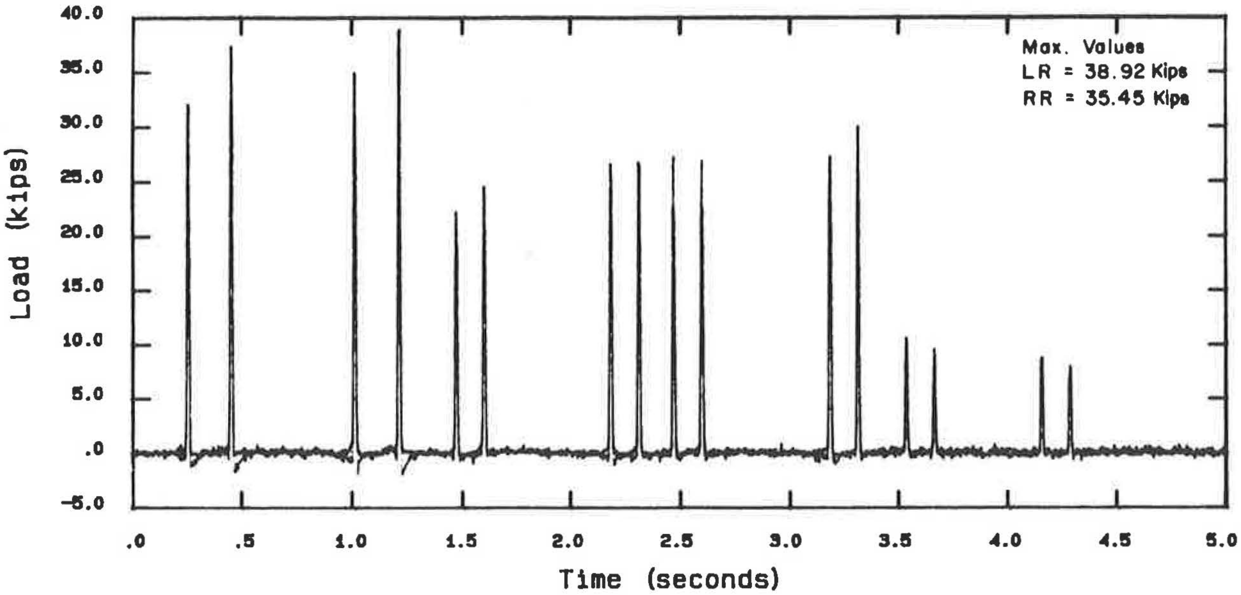


**FIGURE 11 Typical measured load versus time for midspan of second bridge at 30 mph.**

TABLE 2 MAXIMUM MEASURED LOADS AT WHEEL-RAIL INTERFACES

a) Test site #1 - test train #2

| Speed (mph) | Span #S3 Static | Span #S3 Dynamic | Approach Static | Approach Dynamic | Track Static | Track Dynamic |
|---|---|---|---|---|---|---|
| 1 | 31.45 | 34.51 | 31.45 | 34.14 | 31.73 | 35.31 |
| 30 | 31.45 | 36.04 | 31.73 | 40.63 | 31.73 | 38.43 |
| 50 | 31.73 | 36.00 | 31.73 | 50.93 | 31.73 | 43.60 |

b) Test site #2 - test train #2

| Speed (mph) | Span #S2 Static | Span #S2 Dynamic | Approach Static | Approach Dynamic | Track Static | Track Dynamic |
|---|---|---|---|---|---|---|
| 1 | 31.73 | 34.62 | 31.73 | 36.43 | 31.73 | 35.30 |
| 30 | 31.45 | 40.17 | 31.73 | 41.26 | 31.45 | 38.43 |
| 50 | 31.73 | 34.57 | 31.45 | 40.00 | 31.73 | 39.21 |

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 that the chords behaved as simply supported beams. It may be noted that for the open-deck bridge span the value of the DDFs increases with an increase in the speed (i.e., Dd/Dsc varies from 2.1 to 2.7 and Dd/Dcr from 1.0 to 1.3 at speeds of 50 mph). On the other hand, speeds of up to 50 mph did not appear to have any effect on the ballast-deck bridge span for which average values of Dd/Dsc = 1.7 and Dd/Dcr = 1.0 were obtained.

## Accelerations

A typical output of measured acceleration versus time at the midspan of the second bridge for Test Train 2 at 30 mph is shown in Figure 16. It was noted that the range of the measured accelerations widened as the speed increased. For the ballast-deck bridge, the maximum acceleration ranged from +10.08 *g* to −7.00 *g*, but, unfortunately, for the open deck-bridge at 20 mph and beyond, the range exceeded the measurement limit of the instrumentation, which was set at +10.08 *g*.

## Damping in Bridge Spans

The logarithmic decrement technique was applied 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 9.8 percent for ballast-deck span S3 and 6.2 percent for open-deck span S2.

## SUMMARY

Analysis of the data obtained from the tests at the two sites led to the following conclusions:

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

2. The load-deflection behavior of the bridge spans was found to be fairly linear, in contrast with the nonlinear behavior 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 (DLFs) were found to increase in value with increasing train

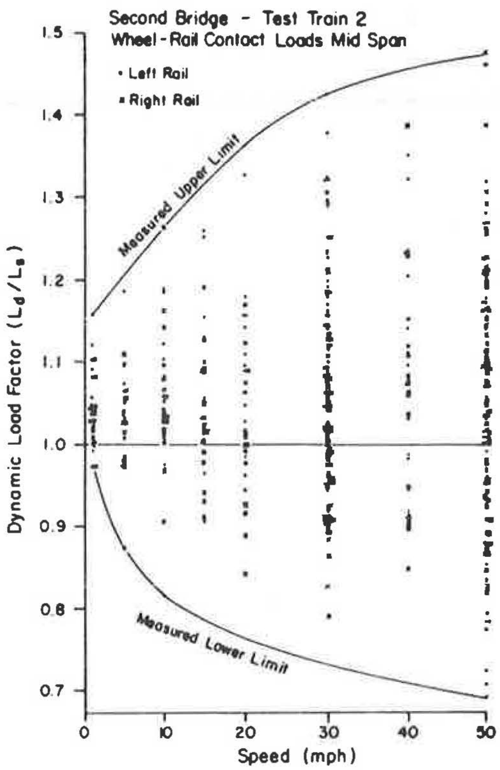


**FIGURE 12 Effect of speed on dynamic load factor.**

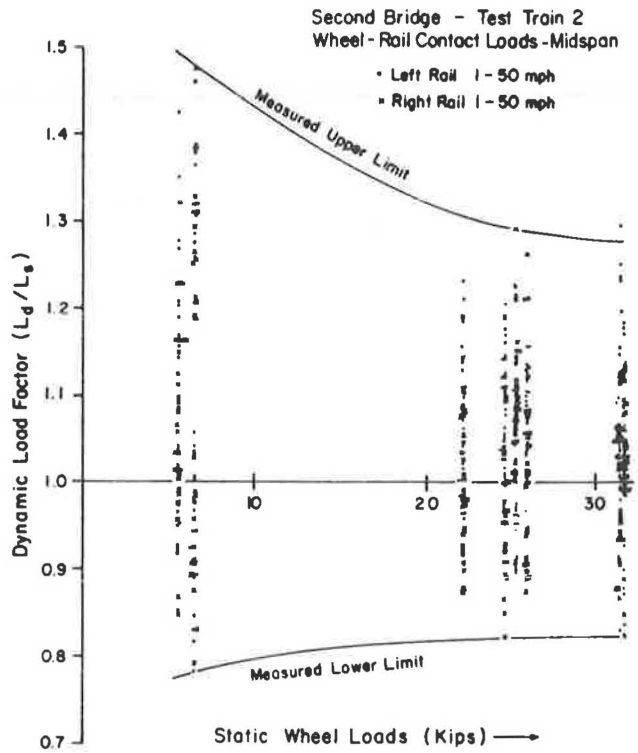


**FIGURE 13 Effect of static wheel load on dynamic load factor.**

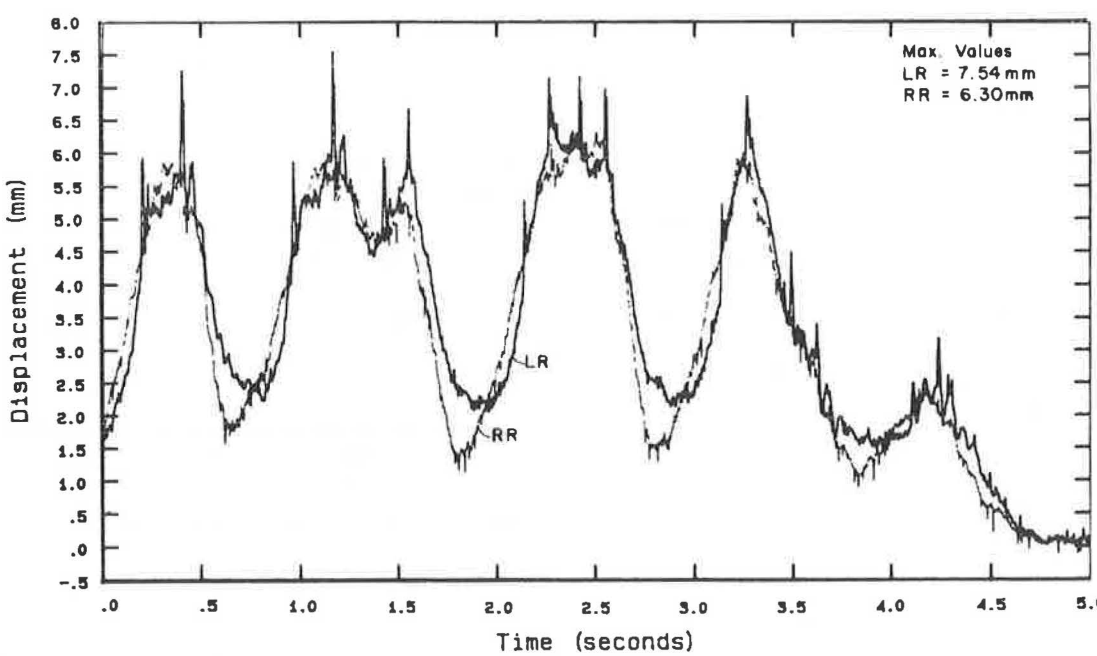


**FIGURE 14 Typical vertical displacement versus time for midspan of second bridge at 30 mph.**

TABLE 3 MAXIMUM MEASURED VERTICAL DISPLACEMENTS

a) Test site #1 - test train #2

| Speed (mph) | Span #3 L Rail | Span #3 R Rail | Span #2 L Rail | Span #2 R Rail | Track L Rail | Track R Rail |
|---|---|---|---|---|---|---|
| 1 | 5.22 | 4.03 | - | 4.10 | 11.92 | 10.14 |
| 30 | 5.46 | 4.00 | - | 4.14 | 12.43 | 9.89 |
| 50 | 5.39 | 4.17 | - | 4.71 | 13.31 | 11.12 |

b) Test site #2 - test train #2

| Speed (mph) | Span #2 L Rail | Span #2 R Rail | Approach L Rail | Approach R Rail | Track L Rail | Track R Rail |
|---|---|---|---|---|---|---|
| 1 | 6.29 | 6.36 | 9.77 | 10.02 | 13.13 | 12.13 |
| 30 | 7.54 | 6.43 | 9.45 | 10.16 | 13.87 | 13.12 |
| 50 | 8.11 | 8.32 | 9.80 | 9.71 | 15.66 | 13.58 |

NOTE: Values are in millimeters.

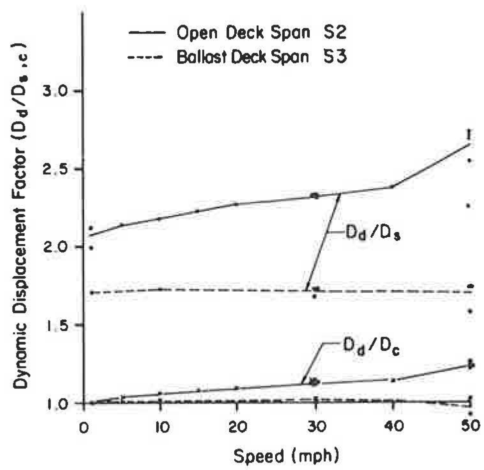


**FIGURE 15 Effect of speed on dynamic displacement factor.**

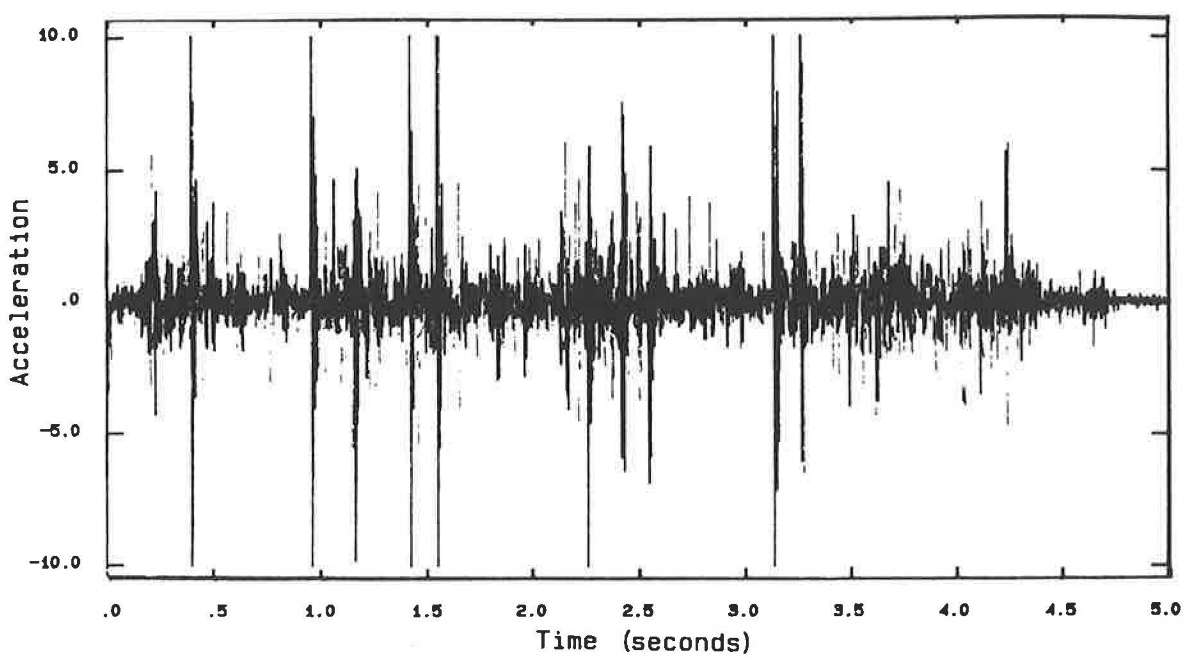


**FIGURE 16 Typical measured acceleration versus time for midspan of second bridge at 30 mph.**

speed. The maximum value of DLF measured was 1.49 at 60 mph. The DLFs were also found to decrease with increasing static wheel loads.

4. For the open-deck span, the dynamic displacement factors (DDFs) increased with increasing speed and had a maximum value of 1.316 over 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 increasing train speed. At speeds above 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 percent more than in the open-deck span.

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