BEHAVIORAL LOAD TESTING OF THE DISRAELI FACILITY

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ABSTRACT

The Disraeli Facility, which was completed in 1960, consists of several overpasses utilizing rolled steel beam construction and a riveted steel plate girder bridge crossing the Red River in Winnipeg, Manitoba. The total length of the facility is approximately 2,320 feet (707 metres). In 1984, the City of Winnipeg commissioned Reid Crowther and Partners Limited to perform a load test on the facility to ascertain the possibility of increasing the maximum gross vehicle weight limit. The tests were performed on three consecutive Sundays, from September 23rd to October 7th, 1984.

Three spans were tested. One normal and one skewed span were selected for the overpasses in order to study possible differences in their behaviour along with the exterior span of a three-span continuous riveted plate girder bridge over the Red River. The test was designed to determine the structural response of the bridges at different load levels, to determine the load distribution characteristics and to investigate dynamic impact values for the test vehicles.

This paper describes the instrumentation layout, data acquisition system, test vehicles and testing procedures. Test results and comparisons with the predicted values utilizing conventional analysis are included.

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INTRODUCTION

The Disraeli facility consists of a series of overpass and bridge structures crossing the C.P.R. mainline and Red River in Winnipeg, Manitoba. An aerial view of the facility is shown in Figure 1. Built in 1959 and 1960, the facility is comprised of a total of 30 spans with an overall length of approximately 2,320 feet (707 metres). Elevation views of the overpass and bridge structures are shown in Figure 2.

![Aerial View of Disraeli Facility](image)

Upgrading of the facility, between 1971 and 1973, consisted of infilling the open grid metal deck grating with concrete, providing a 2-inch (50 mm) asphalt-wearing surface, and strengthening of the superstructure to support the heavier dead loading and to increase the design live loading from AASHTO H20 to HS20.

In response to the upward trend in legal load limits, the facility was load rated in 1978 to determine whether or not the structure could accommodate AASHTO HS25 loading, and the degree of strengthening required to support that loading.

Conclusions contained in the load rating report indicated that the structures would require further strengthening to support HS25 loading.
FIG. 2  Elevation Views of Disraeli Overpass and Bridge
Lacking information concerning the loading history of the facility, coupled with concerns regarding the fatigue life of structures, the report also recommended that a load test be undertaken to confirm load capacity.

In 1984, the City of Winnipeg commissioned Reid Crowther to perform a load test on the facility. The load test was performed on three consecutive weekends in September and October of 1984.

OBJECTIVES

The primary objectives of the load test could be summarized as follows:

1. To determine the actual load distribution characteristics and stress levels in various elements of the structure under known loadings.
2. To determine the structural mechanism in terms of composite action, and the linearity of structural response of main girders under varying loads.
3. To investigate the dynamic response of the structure under varying loadings travelling at various speeds.
4. To measure static deflections under known loadings.
5. To compare measured stresses in the deck system; stringers, floorbeams, channels and grating, with predicted stresses determined in accordance with the AASHTO Bridge Code.

SELECTION OF TEST SPANS

Typical cross-sections of superstructures on the Disraeli overpass and bridge are shown in Figures 3 and 4, respectively.

The rolled beam spans, including the entire overpass and Spans 1 to 4 and 10 of the bridge, are basically the same with the only major variation being in span length and support conditions. For the reader's reference, span numbers are identified by the pier number immediately to the left of that span. Two spans of the overpass, Span 14 and Span 16, were selected for load testing in order to represent both normal and skewed bridge types having rolled beams as the main load carrying members.

The riveted plate girder spans over the Red River consist of a three span continuous haunched girder bridge flanked by two simple spans of the same construction. Span 8 was selected from the bridge as representative of the plate girder spans. The same span was also identified by the 1978 Load Rating Report as requiring strengthening to support the proposed heavier loading.
FIG. 3 Typical Cross Section of Rolled Beam Spans

FIG. 4 Typical Cross Section of Riveted Plate Girder Spans
Other factors considered in the selection of the spans were proximity to electrical power, location of other test spans, and accessibility for instrumentation and monitoring.

TEST VEHICLES

Four gravel truck-trailers were used for test vehicles. The trucks, Figures 5 and 6, available locally, were selected for their close similarity in axle spacing and weights to the AASHTO HS25 design vehicle.

In order to establish the linearity of structural response for various bridge members being monitored, three Trucks A, B and C, with gross vehicle weights of approximately 50 kips (222 KN), 72 kips (320 KN) and 90 kips (400 KN), respectively, were used.

A fourth vehicle, Truck D, loaded almost identically to Truck C was used in order to examine the validity of load superposition. Test results for a particular member under multiple lane loading were obtained by combining single test vehicle results for each lane separately. To verify the accuracy of this method, results obtained by adding Truck C data in adjacent lanes were compared to results obtained by placing Trucks C and D in adjacent lanes simultaneously.

FIG. 5 Test Vehicle at Pre-marked Location on Deck
FIG. 6  Comparison of Design and Test Vehicles
Gravel was selected as the load material, since it was a stable medium which would maintain axle load distribution between weigh scales and the test site.

INSTRUMENTATION

Extensive measurements were recorded during various tests on the overpass and the bridge. Measured quantities included steel and concrete strains, girder reactions and deflections. A total of 243 electric resistance strain gauges were used to measure the strain electronically. Each active gauge was connected to an identical "dummy" gauge attached to a separate steel element independent from the bridge structure. These "dummy" gauges were located close to the active gauges to compensate for temperature effects during the test. They were safely secured along with lead wires and connectors beneath the main girders ready for direct connection to the main cables of the data acquisition system during each load test, as shown in Figure 7.

All gauges were located at predicted locations of maximum positive and negative moments. Three gauges were used on each member tested to determine the strain profile at each given location. For the main girders and floor beams, gauges were attached to the top and bottom flanges at the mid-height of the flange thickness, while the third gauge was located at the mid-height of the section, as shown in Figure 8. The

FIG. 7 Dummy Gauges and Lead Wires on the Bridge
decision to locate the gauges on the sides of the flanges was made to avoid the possible strain concentration effect due to the presence of bolts in the cover plates. For the channels, the top and bottom gauges were located on the interior surface of the flanges as shown in Figure 8.

For Span 14 of the overpass, a total of 99 gauges were used as follows: 36 gauges for the girders, 60 for the channels and 3 to measure the concrete strains at the bottom surface of the concrete infilled metal deck grating. For the skewed span of the overpass, Span 16, a total of 36 gauges were used to monitor only the main girders. A total of 109 gauges were used on Span 8 of the bridge as follows: 24 for the main girders, 12 for the floor beams, 18 for the stringers and 54 for the channels.

The gauges used for the steel members were Series CEA self temperature compensating gauges supplied by Measurements Group Inc. They were supplied with a fully encapsulated grid and exposed copper-coated integral solder tabs for direct solder of the heavy lead wires. The gauges used for the concrete were 2-inch long EA-06-20, CBW-120 type. All gauges were installed and coated against moisture using M-Coat G protective coating system supplied by Intertechnology Inc.

For static loading tests, the readings were recorded electronically using a 3490A digital multimeter connected to a 3495A scanner for a total of 120 channels. The data were stored in a HP9825 desktop computer for future processing and analysis.
For dynamic tests, a total of four gauges on each test span of the overpass and the bridge were connected to a Nicolet 2090/204A digital storage oscilloscope with scanning capability of 20 million readings per second. Each gauge was connected separately to a strain gauge conditioner for amplification, filtering and bridge completion of the electric circuit. The data were stored on floppy disks for future display and plotting. The device also provides the immediate display of the strain under the dynamic load.

The entire data acquisition system, including the x-y plotter, was mounted in a rented van which was driven to the bridge site during installation of the gauges and during testing of the overpass and the bridge. Installation of the gauges, calibrations and checking the connecting wires for the three spans tested required seven weeks. A scissor lift scaffolding was used to install the gauges and the connection wires for the overpass. However, scaffolding had to be built and suspended beneath the bridge to enable instrumentation of the bridge span over the river.

Dead load and live load reactions of one main girder of the overpass were measured using a flat type load cell, Model FL-200C of 200 Kip (890 KN) capacity. Deflection at the maximum moment location of the overpass was measured by a dial gauge supported on a steel scaffolding. The deflections were also measured for different truck positions using survey instruments.

A total of four mechanical strain gauges, with 20 inch gauge length, were installed on the tension flanges of the maximum negative and positive moment locations of the main girders on the overpass and the bridge. The gauges are self-contained mechanical instruments recording strain-induced deflections of a stylus on a rotating target which advances automatically for permanent documentation of individual events. The devices are used to measure the strain induced under normal traffic conditions to obtain data for evaluation of the fatigue life of the bridge. The targets can be removed after a specified time and strain data can be obtained by analyzing the scratch data on the target using a calibrated microscope.

TEST PROCEDURE

Load tests for the three spans were conducted on three consecutive Sundays to minimize the effects of the traffic closures. The actual test for each span was delayed until 11:00 a.m. to avoid large fluctuations in the temperature and to ensure the stability of the data acquisition system strain gauges. Communications between the bridge engineer, controlling truck movements to the locations pre-marked on the bridge deck for each test, Figure 5, and the data acquisition operator, were simplified by using a two-way radio system. Before each test, for each truck type, the readings of the strain gauges were "zeroed" initially to minimize the temperature, wind and vibration effects on the bridge.
After moving a truck into position, time was allowed to ensure the bridge had stopped vibrating before strain gauge data were read and deflections recorded. Approximately four hours were required for static testing on each span. After completing the static tests, four gauges were connected to the dynamic data acquisition system to record the strain response under various truck loads using different lanes. Dynamic testing was performed at two speed levels of 10 mph (16 kph) and 40 mph (64 kph). A 2" x 10" (38 mm x 235 mm) wood plank was used to simulate surface roughness and possible presence of a bump. Dynamic testing required approximately three hours to complete.

During testing of the overpass, it was convenient to park the test van directly beneath the test spans. However, in order to be close to the instrumentation, the test van was parked over Pier 8 in the curb lane during static tests on the bridge over the river. During the dynamic testing of the bridge, the data acquisition equipment was placed on the sidewalk and the van removed to allow use of all four lanes of the bridge.

At the end of the day, the static readings were tabulated and the dynamic responses were plotted using an x-y plotter. A typical result for the dynamic response is given in Figure 9.

![Graph of strain vs. time showing static and dynamic responses](image)

**FIG. 9 Typical Dynamic Test Results**
TEST RESULTS

Linearity of Structural Response

Stresses derived from measured strains in the rolled beams and girders under each of the three different vehicle loadings are shown in Figures 10 and 11 respectively. The results clearly indicate that the structural response is nonlinear under increasing vehicle load.

Stresses measured in the first interior girder on Span 14 are plotted against increasing loads, as shown in Figure 10. By examining those results, it would appear that, under Truck A and B loading, the girder stresses increase linearly and appear to enjoy some composite action with the deck system as evidenced by the location of the neutral axis. Under Truck C loading, the location of the neutral axis shifts and the stresses "veer off" their predicted values. It appears that, under the heavier loads, the girders almost behave completely noncompositely, likely due to twisting of the channel sills supporting the deck.

Stresses measured in a riveted plate girder on Span 8 produced similar results, although the stresses recorded were unexpected. By examining Figure 11, the same general trend seen on Span 14 toward noncomposite action is illustrated. However, it was interesting to note that the girder stresses in the negative moment region are all positive. At this stage, we suspect that light wind loading on the span produced tensile stresses in the sides of the flanges. Since the strain gauges were mounted on the sides of the flanges rather than being centred beneath the web, it is possible that they were influenced by this biaxial loading.

Test results of the skew span, Span 16, were virtually identical to Span 14 results.

It is important to note that the results illustrated here are based on data recorded from single lane loading. It is therefore possible that a similar response to the Truck C loading could be obtained by positioning two trucks, A or B, in adjacent lanes.

We conclude that the girders on both the bridge and overpass behave noncompositely under the heavier vehicle loads.

Live Load Distribution Factors

Live load distribution factors were calculated for each girder, under the various loadings, positioned in accordance with AASHTO to cause the maximum effect in the tested member.
FIG. 10  Linearity of Structural Response Disraeli Overpass - Span 14
FIG. 11  Linearity of Structural Response Disraeli Bridge - Span 8
For the overpass, Span 14 distribution factors are plotted against AASHTO values and number of lanes loaded in Figure 12. The values shown are actual distribution factors calculated from the load test data and have not been reduced in accordance with AASHTO for 3-lane or 4-lane load cases. The results obtained from Span 14 indicate that the exterior girder takes slightly more loading than the predicted values while the interior girders are lower. It is also very clear that the distribution factors in both positive and negative moment regions are almost the same.

The distribution factors for Span 16, plotted in Figure 13, are slightly higher for the exterior girder as compared to Span 14 results.

For the bridge, the live load distribution factors calculated for Span 8 are plotted on Figure 14. The results indicate that both the exterior and interior girders behave similarly to the AASHTO prediction with the maximum values slightly higher than predicted.

Since the spans tested are typical of adjacent spans in the overpass and bridge, the obtained distribution factors could be applied for the entire bridge facility.

The maximum distribution factors derived from the load test are summarized in Table 1 for comparison. Values obtained under Truck B loading are also presented.

<table>
<thead>
<tr>
<th>Location</th>
<th>Truck B</th>
<th>Truck C</th>
<th>AASHTO Predicted</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Bridge:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- interior girder</td>
<td>2.71</td>
<td>2.73</td>
<td>2.54</td>
</tr>
<tr>
<td>- exterior girder</td>
<td>1.74</td>
<td>1.83</td>
<td>1.56</td>
</tr>
<tr>
<td>2. Overpass:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- interior girder</td>
<td>1.91</td>
<td>1.94</td>
<td>2.00</td>
</tr>
<tr>
<td>- exterior girder</td>
<td>1.19</td>
<td>1.17</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Deck System

The 1978 Load Rating Report predicted that the deck grating and channel sills on both the bridge and overpass would not require strengthening under the proposed loadings. It was uncertain, however, whether or not the deck system behaved as a flat plate or rather as a membrane. For this reason, it was recommended that the capacity of the deck system be confirmed by load test.
FIG. 12  Live Load Distribution Factors on Rolled Beam Spans (Truck C Loading)
FIG. 13  Live Load Distribution Factors on Rolled Beam Spans (Skewed) (Truck C Loading)
FIG. 14  Live Load Distribution Factors on Plate Girder Spans (Truck C Loading)
The maximum live load stresses in the channels and grating determined by the load test, are compared to predicted values using the AASHTO Bridge Code and a finite element analysis in Table 2. By examining the Table, it is clear that the live load stresses in the deck grating and channels are approximately 40 to 50% lower than the predicted values.

**TABLE 2**

MAXIMUM LIVE LOAD STRESSES IN THE DECK SYSTEM - USING TRUCK C LOADING

<table>
<thead>
<tr>
<th>Location</th>
<th>Measured</th>
<th>Predicted</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AASHTO</td>
<td>Finite Element</td>
<td></td>
</tr>
<tr>
<td>Deck Grating</td>
<td>3.50 ksi (24.1 MPa)</td>
<td>7.25 ksi (50.0 MPa)</td>
<td>6.33 ksi (43.7 MPa)</td>
</tr>
<tr>
<td>Channels (CI0)</td>
<td>5.10 ksi (35.2 MPa)</td>
<td>9.37 ksi (64.6 MPa)</td>
<td>7.34 ksi (50.6 MPa)</td>
</tr>
<tr>
<td>Channels (C9)</td>
<td>4.5 ksi (31.0 MPa)</td>
<td>10.53 ksi (72.6 MPa)</td>
<td>7.41 ksi (51.1 MPa)</td>
</tr>
</tbody>
</table>

The 1978 Load Rating Report indicated that the stringers and floorbeams were overstressed when they were analyzed using AASHTO for HS25 loading. The predicted stresses in the stringers and floorbeams using finite element analysis were 35% and 40% respectively lower than the AASHTO values. In both cases, the floorbeams were found to be overstressed. The predicted stresses in the stringers and floorbeams in accordance with the AASHTO Method and by Finite Element Analysis for Truck C loading are compared with the measured stresses in Table 3.

By examining Table 3, it is evident that the induced stresses in the stringers and floorbeams are much less than the values predicted by conventional analysis.

**TABLE 3**

STRINGERS AND FLOORBEAMS

MAXIMUM LIVE LOAD STRESSES - TRUCK C LOADING

<table>
<thead>
<tr>
<th>Location</th>
<th>Measured</th>
<th>Predicted</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AASHTO</td>
<td>Finite Element</td>
<td></td>
</tr>
<tr>
<td>Stringers</td>
<td>4.4 ksi (30.4 MPa)</td>
<td>12.3 ksi (84.8 MPa)</td>
<td>7.87 ksi (54.3 MPa)</td>
</tr>
<tr>
<td>Floorbeams</td>
<td>6.35 ksi (43.8 MPa)</td>
<td>14.61 ksi (100.8 MPa)</td>
<td>10.23 ksi (70.6 MPa)</td>
</tr>
<tr>
<td>W21 x 68</td>
<td>5.0 ksi (34.5 MPa)</td>
<td>15.13 ksi (104.3 MPa)</td>
<td>10.59 ksi (73.0 MPa)</td>
</tr>
</tbody>
</table>
Miscellaneous Results

The dynamic response of the structure was investigated under Truck A and Truck C loadings moving at two speed levels of 10 mph (16 Kph) and 40 mph (Kph). Results obtained suggested impact values as high as 78% with an imposed 1-1/2 inch (38 mm) bump under Truck A loadings. However, it should be noted that the dynamic testing was performed mainly to obtain a "feel" for impact values induced by the test trucks rather than the absolute values since these values would vary for different vehicles depending on such aspects as suspension system, weights, number of axles and the span being tested. Typical values of the impact factors are shown in Figure 15. The results suggested that, for a given vehicle, the impact values increased with decreasing load and speed.

![Graph showing impact values for Truck A and Truck C at different truck speeds.](image)

FIG. 15 Positive Moment Impact Factors on Span 14 of Overpass

The measured static deflections recorded on Span 14, using a mechanical dial gauge, indicate a maximum measured deflection corresponding to L/1400 with Trucks C and D positioned side by side. This value compares favourably with the recommended AASHTO limit of L/1000 for bridges with pedestrian traffic.

The dead and live load reactions at Pier 14 on the first interior girder were measured, utilizing a load cell. Dead load reactions were found to be approximately 36% higher than the predicted value, while the live load reactions were between 20% and 30% lower than predicted values.
SUMMARY AND CONCLUSIONS

To study the possibility of increasing the maximum gross vehicle weight limit, two spans of several overpasses using rolled steel beam construction and one span of continuous haunched riveted steel plate girder bridge were subjected to static and dynamic load testing. Four gravel truck-trailers of different weights were used as test vehicles. The measured quantities included steel and concrete strains, girder reactions and deflections. The instrumentation was located at predicted critical locations. The tests were performed on three consecutive Sundays to minimize the effects of the traffic closures.

Based on the measured values, the linearity of the structural response, live load distribution factors and the impact level were determined and compared to the predicted values using conventional structural analysis and AASHTO specifications.

Conclusions based on the up-to-date findings may be summarized as follows:

1. The structures exhibit equal or better live load distribution characteristics than predicted values by AASHTO specifications.
2. The main load carrying members, girders and rolled beams, act non-compositely with the deck system under the heavier loadings.
3. Live load stresses in the deck system; grating, channels, stringers and floorbeams; were found to be lower than the predicted values by conventional and finite element computer analyses.
4. For a given vehicle, the impact factor decreases with increasing weight and speed.

ACKNOWLEDGEMENTS

This load test was carried out for the City of Winnipeg, Streets and Transportation Department, by Reid Crowther and Partners Limited. Instrumentation, installation and data collection during the tests were carried out by the University of Manitoba, Department of Civil Engineering.