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SEISMIC RESISTANCE TESTS ON AN OPEN-DECK STEEL BRIDGE

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Summary

Lateral-resistance tests of five 62-foot, open-deck (OD) deck-plate-girder (DPG) steel spans indicate that these spans had considerable resistance to the types of movement that could be expected during an earthquake. These tests were conducted by Transportation Technology Center, Inc. on the Norfolk Southern Vine Street Terminal Connection Viaduct in Cincinnati, Ohio, in October 1998. The purpose was to quantify resistance of a typical DPG-OD railroad bridge to ground motions and to determine the contribution of the rail to the total resistance. Three major contributing factors to the resistance of this bridge are the anchor bolts, friction, and continuity of the track structure. Test results are summarized as follows:

- Peak lateral resistance of a span with rails cut at both ends was about three times the dead weight of the span.
- The rails attached to the approach embankment contributed about 20 kips of additional resistance to lateral movement of one end of a span.
- The rails attached to an adjacent span contributed more than 100 kips of additional resistance to lateral movement of one end of a span.
- The resistance to lateral movement provided by frictional and locking forces was slightly more than the dead weight of the span.
- These results show that the lateral resistance of this type of railroad bridge exceeds some of the most severe requirements used in seismic design.

Under certain conditions, the resistance of the approach embankment could be reduced by vertical uplift or liquefaction. These findings are important for the assessment, design, and possible retrofit of railway bridges for earthquake resistance. This work was performed as part of the Association of American Railroads' Strategic Research Initiative on railroad bridges.

Suggested Distribution:
- ☐ Maintenance of Way
- ☐ Bridges & Roadway
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INTRODUCTION AND CONCLUSIONS

In order to assess the ability of railroad bridges to withstand earthquakes, information is needed to quantify the resistance of existing bridges to horizontal ground motions. With this information, decisions can be made about the need to retrofit a bridge, and if so, what sort of retrofit might be appropriate. A previous test[1] on a ballast-deck (BD) through-plate-girder (TPG) span showed that the track structure provided significant resistance to lateral movement for that ballast-deck bridge. Based on the previous test, many railroad bridges of that type were deemed to have sufficient earthquake resistance so that retrofits were not necessary.

The purpose of tests reported here was to quantify the earthquake resistance of a typical open deck (OD), deck-plate-girder (DPG) railroad bridge to ground motions that may result from an earthquake and to determine the contribution of rail to the total resistance. For this open-deck bridge, the contribution of the track structure was not as great as for the BD-TPG bridge, as the rails provide the sole connection between the bridge and the approach embankment. Based on the dead weight of the spans, however, these open-deck spans had greater capacity to resist horizontal ground movements than the ballast deck spans tested earlier.

As three identical 62-foot spans were pushed, lateral resistance was measured. In order to quantify the additional resistance added by the rails, tests were conducted both with rails attached and with rails cut between spans.

The resistance of all spans to lateral forces was considerable:

- Laterally with rail cut — three times the dead weight of the span.
- Laterally with rails attached to the approach embankment — about 20 kips additional resistance due to track structure.
- Laterally with rails attached to the adjacent span — about 100 kips additional resistance due to track structure.

The resistance to horizontal movement was provided primarily by three components: anchor bolts, frictional and locking forces, and rail connected to the approach embankment or adjacent spans.

TEST PROGRAM

Prior to testing, the portion of the bridge to be tested was thoroughly inspected. Particular attention was given to the condition of rails, ties and their anchoring, and span bearings. The rail joints also were tightened. After each phase of testing, the inspections were repeated and any damage or movement in the components was recorded.

The test program involved three lateral pushes and two longitudinal pushes, as shown in Exhibit 1. There were a total of seven identical spans at the west end of the viaduct. Note that only the westernmost span (span 16) was an end span. All other spans were intermediate spans.

The details of the lateral tests are as follows:

- Span 16 pushed from the west abutment with rails attached.
- Span 17 pushed from Pier 16 with rails attached.
- Span 18 pushed from Pier 17 with rails cut at both ends.

Steel reaction frames were installed on the abutment or pier at the west end of the span to be pushed. A 200-ton capacity jack powered by a 10,000-psi pump was mounted between the span and the reaction frame. Lateral thrust was incrementally applied to the span. Measurements were taken at each location.

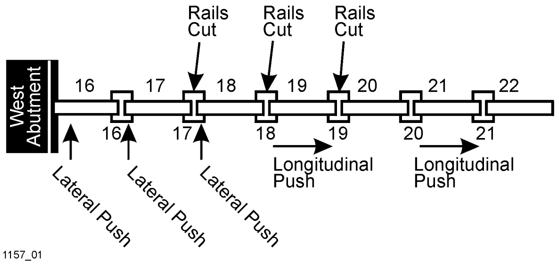

Exhibit 1. Overall Plan of Lateral and Longitudinal Tests

LATERAL RESISTANCE TEST RESULTS

Three lateral-resistance tests were performed by pushing one end of a span with a hydraulic actuator, reacting against the pier or abutment. Exhibit 2 shows the relative displacement between spans after completion of a lateral test with rails cut. For all tests, the walkway timbers were cut between spans.

One end of each 61-foot, 9-inch-long DPG span was pushed laterally over the abutment or

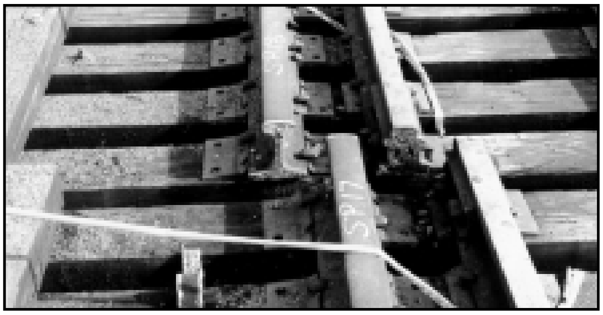

Exhibit 2. Lateral Displacement with Rails Cut

pier. Anchor bolts were sheared in two of the three positions. The baseline case was at span 18, an intermediate span, with the rails cut at both ends. Lateral resistance was 192 kips, or 3.0 g. All four anchor bolts were sheared.

The end span was tested at the west abutment with the rail continuous onto the approach embankment. The total resistance was about 214 kips (3.4 g). Anchor bolts were sheared. Track on the bridge deck near the abutment stayed with the span as it was pushed laterally and showed no signs of distress. Track on the approach embankment exhibited ties shifting laterally in the ballast section. The ballast section was generally fouled, with some vegetation present. There was no damage noted to the ties, tie plates, or spikes, all of which were in generally good condition.

The force-displacement results for the lateral resistance tests are summarized in Exhibit 3. Note that the trace for the span with rails attached at the abutment shows a peak lateral force about 20 kips higher than that for the span with rails cut. The span with the rails cut, however, sustained a relatively high lateral resistance to a displacement of more than one inch. The difference might be due to variations in the fit or the amount of corrosion of the anchor bolts. The flat portions of the traces beyond 3 inches of displacement show the amount of resistance due to friction. The estimated weight of the span was 126 kips total, or 63 kips per end reaction. The amount provided by the forces due to friction and locking of components was at least as much as the end reaction. In an earthquake the frictional resistance might be somewhat less if there is significant vertical ground motion which could result in decreased end reaction.

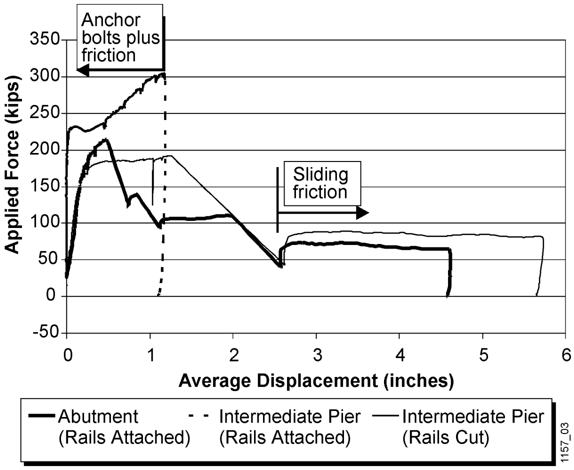

Exhibit 3. Lateral-Resistance Test Results

Track panel push test results suggest that the resistance provided by the track structure that continues over the abutment should provide about 11 kips of lateral resistance for a movement of about a quarter inch.[2] Results from this test show an additional 22 kips of resistance provided by the abutment. The difference may be due to the larger displacement (about a half-inch for peak resistance), additional resistance provided by the guard rails, and variability in span resistances provided by anchor bolts and friction. The amount of resistance provided by the track structure should be independent of span length. It may depend on the presence of guardrails continuous onto the approach embankment.

At another intermediate span, the rails were left intact and continued to the adjacent span. Here, the total resistance was at least 305 kips (4.8 g). This was a 114 kip or 59 percent increase over the base case. Application of force was limited to 305 kips due to incipient failure of the deteriorated surface concrete of the pier. Anchor bolts were not sheared. Maximum displacement was 1.2 inches. The results indicate that rails can add significantly to total lateral resistance when securely fastened to an adjacent span. The failure plane at the bearings was not generally in the same location. In some cases the failure was between the bearing and the abutment, in other cases

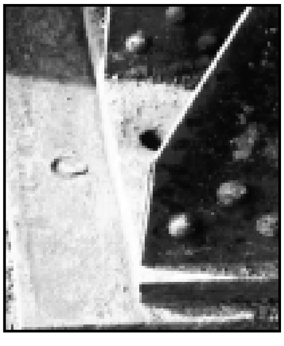

Exhibit 4. Failed Bearing at West Abutment

it was between plates of the bearing assembly. Exhibit 4 shows the displaced bearing plates at the west abutment after completion of the lateral resistance test. Note the relative position of the bolt holes and the sheared anchor bolt.

COMPARISON TO PREVIOUS TEST

Compared to the ballast-deck through-plate-girder bridge tested previously,[1] this bridge had lower lateral resistance in terms of the lateral force required to move it. But it had higher lateral resistance in terms of the lateral acceleration required. This is due to the lighter weight of an open-deck structure. The resistance in terms of acceleration is particularly relevant for determining earthquake resistance, as earthquake loading is typically expressed in terms of ground acceleration.

Exhibit 5 summarizes and compares lateral-resistance test results from this test and the previous test. As expected, the contribution of the track structure was much less on the open-deck span, as the rails were the sole element that continued between the bridge and the approach embankment. In the case of the ballast-deck span, significant resistance was provided by the ballast pan. The flat-plate bearings on the open-deck spans reached a state of sliding at displacements beyond about 3 inches. In the previous tests, a state of sliding was not reached due to rocking and tipping of the high seat bearings.

Specification	Bridge Type	
	BD-TPG	OD-DPG
Span Length	75 feet	62 feet
End Reaction	160 kips	63 kips
Peak Lateral Resistance (Rails Cut)	264 kips (1.65 g)	192 kips (3.0 g)
Peak Lateral Resistance (Rails Attached to Approach)	385 kips (2.4 g)	214 kips (3.4 g)
Contribution of Track Structure	121 kips	22 kips

Exhibit 5. Comparison of Lateral-Resistance Test Results

BRIDGE DESCRIPTION

The Norfolk Southern Vine Street Terminal Connection Viaduct was a 41-span elevated track structure in Cincinnati, Ohio. The west end of the viaduct had seven identical 62-foot deck-plate-girder spans of riveted steel construction. They rested on flat-plate type bearings and were supported on concrete piers.

Track on the bridge and approach embankment was of conventional construction. Rail was 132 lb/yd jointed rail, with 100 lb/yd inner guardrails (also jointed). The guardrails extended 16 feet beyond the back wall of the bridge. Rail was fastened to timber ties with tie plates and cut spikes. Bridge deck ties were 8 inches wide by 13 inches deep, on 14-inch centers. Rail was anchored to approximately every fourth tie on the bridge deck. There were few rail anchors on the approach embankment.

The results of these tests will be shared with American Railway Engineering and Maintenance of Way Association committee 9 — Seismic Design for Railway Structures, to be used in the development of manual provisions on design and retrofit of railway bridges for earthquake resistance.

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2. Li, D., and Shust, W., "Investigation of Lateral Track Strength and Track Panel Shift Using AAR's Track Loading Vehicle," Association of American Railroads Research Report R-917, December 1997.

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