TECHNOLOGY DIGEST

TD-07-026
©SEPTEMBER 2007

TIMELY TECHNOLOGY TRANSFER

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Continuous Welded Rail Restraint on an Open-Deck Girder Bridge

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Summary

To aid in development of guidelines for continuous welded rail (CWR) on open-deck bridges, the Transportation Technology Center, Inc. carried out testing in January 2007 on the 121-foot deck plate girder bridge installed at the Federal Railroad Administration's Facility for Accelerated Service Testing in Pueblo, Colorado.

Based partially on previous tests,[1,2,3,4] a model was developed to predict behavior of CWR under various anchoring conditions on bridges. The model will be used along with the results from revenue service testing in support of the recommendation of guidelines for CWR on long bridges.

Test conclusions include:

- High strain rate applications, such as the instantaneous release of tension during a rail break under thermal tension, result in about 50-percent lower resistance to longitudinal movement than, for example, gradual strain rate applications that would be applied by thermal expansion and contraction of a bridge girder.
- On smooth top girders, longitudinal restraint is weakest at the tie-to-girder interface. For the bridge configuration tested, the restraint realized between the tie and girder is about 1/4 of that between the rail and tie.
- While rail anchors exhibit considerable longitudinal resistance when tested per AREMA guidelines, longitudinal restraint of CWR on open-deck bridges may be considerably less, by a factor of 2 or more because:
  - AREMA testing is conducted under gradual strain rate force application, and
  - a weaker failure plane may exist such as the interface between deck and smooth top girder.

As part of the Association of American Railroads' (AAR) effort to reduce the stress state of the railroad, the elimination of rail joints on bridges has been shown to significantly reduce impact, reduce costs, and extend bridge life.[5,6] Expansion joints may be necessary in some cases to limit rail gap during a rail break and to minimize longitudinal rail forces on bridge approaches, where buckling resistance is often poor. Some bridge engineers try to limit a broken rail gap in CWR to 3 inches. There is no corresponding generally accepted limit for a broken rail gap in open track.

Conclusions and results from this study are being used to calibrate and enhance the CWR model and further develop guidelines for CWR management on open-deck bridges. This study was conducted under the AAR's Strategic Research Initiatives Program.

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INTRODUCTION AND OBJECTIVES

With more than half of steel railroad bridges having open decks, it is important to understand interaction between CWR and bridges. Potential problems include track buckling on bridge approaches, excessive rail gap due to a broken rail, and excessive forces in bridge components. To aid in the refinement of a model to predict CWR performance, Transportation Technology Center, Inc. carried out testing in January 2007 on the 121-foot deck-plate girder bridge installed at the Facility for Accelerated Service Testing (FAST).

As part of the AAR's effort to reduce the stress state of the railroad, elimination of rail joints on bridges has been shown to significantly reduce impact and extend bridge life.[5,6] In order to remove expansion joints from long, open-deck bridges, better understanding and prediction capabilities are needed to deal with thermal effects of CWR.

Handling of CWR on open-deck bridges is a matter of balancing conflicting issues. If rail expansion joints are eliminated, there might be increased potential for track buckling on approaches, particularly near bridge expansion bearings. And the rail gap caused by a broken rail might be too large, while the rail induces high longitudinal forces into the bridge. Some bridge engineers try to limit a broken rail gap in CWR to 3 inches. There is no corresponding generally accepted limit for a broken rail gap in open track.

Based partially on previous tests,[1,2,3,4] a model was developed to predict behavior of CWR under various anchoring conditions on bridges. The model will be used along with results from revenue service testing in support of guidelines for CWR on long, open-deck bridges.

Testing was completed to quantify the force-displacement relationship between rails and girders to calibrate the analytical CWR model under thermal loads. Two types of loads were considered:

- When a rail breaks, all internal thermal rail force (tension) is suddenly released in the form of either a gap between broken rail ends or restrained by fasteners and ties that transfer some portion of the load to the superstructure. This creates an impulsive or high-strain rate application of force.
- When spans expand and contract thermally, force is imparted to the rail through the deck and fasteners. This is a gradual strain rate application of force.

Test conclusions include:

- High strain rate applications, such as a rail break under thermal tension, result in about 50 percent lower longitudinal restraint values than, for example, gradual strain applications that would be applied by thermal expansion and contraction of a bridge girder.
- On smooth top girders, longitudinal restraint is weakest at the tie to girder interface. For the bridge configuration tested, the restraint realized between the tie and girder is about 1/4 of that between the rail and tie
- While rail anchors exhibit considerable longitudinal resistance when tested per AREMA Chapter 5 guidelines, longitudinal restraint of CWR on open-deck bridges may be considerably less.
    - AREMA guidelines suggest testing under gradual strain rate force application.
    - A weaker failure plane may exist such as the interface between deck and smooth top girder.
- Average rail-to-deck longitudinal restraint values recorded should be reasonable for modeling similar structures:
    - 36 pounds/inch for gradual strain rate applications.
    - 19 pounds/inch for high strain rate applications.

Additional testing is under way for a 400-foot open-deck bridge with elastic rail fasteners.

Bridge Description

The steel bridge at FAST is of welded girder construction with an open-deck attached to smooth-top girders. The 65-foot west span nearest the rail cut has spring clips. The 55.5-foot east span has hook-bolts. Every tie is box anchored on the bridge deck and for at least 500 feet on each approach. (See references 4 and 6 for detailed descriptions of the fasteners used.)

Results for a bridge with a deck attached to a girder that has riveted cover plates or other details providing a stronger deck-to-girder interface would likely be significantly different.

Test Procedure

Testing consisted of cutting the rail while under thermal tension and measuring forces and displacements before and after the cut. Forces and displacements were again recorded after pulling the rail back into place with a rail tensor.

Rail cut tests were carried out near the west abutment of the steel bridge at FAST. With the outside rail in about 145,000 pounds of tension (55°F below neutral temperature), the rail was cut. Rail force was measured before and after the rail cut using a series of longitudinal force strain gages applied to the rail at a number of positions (Figure 1). Rail-to-tie and tie-to-girder displacement was measured after the rail had been cut. Girder displacements were measured using standard survey equipment. Track longitudinal resistance was then estimated from these measurements.

A sample of rail anchors was removed from the affected rail and slip tested in the laboratory per AREMA Chapter 5 guidelines (Figure 2) before testing began.[7] None of the anchors removed from the bridge met guidelines for new fasteners. Based on these results, all of the rail anchors were replaced on the inside rail to simulate a new condition. Rail anchors on the outside rail were left intact to simulate typical service conditions.

Figure 1. Rail Force Measurement Circuit

Figure 2. Rail Anchor Test Setup

RESULTS

Figure 3 exhibits the change in longitudinal rail force along the bridge, starting from the west abutment (closest to the rail cut) at zero feet. The greatest change is seen at the cut, where force is reduced by the amount of tension in the rail. The change in force is due to the longitudinal restraint of the rail to the bridge girder. The high linearity indicates that the longitudinal restraint is fairly constant across the bridge, with the slope of the linear fit indicating the longitudinal restraint value.

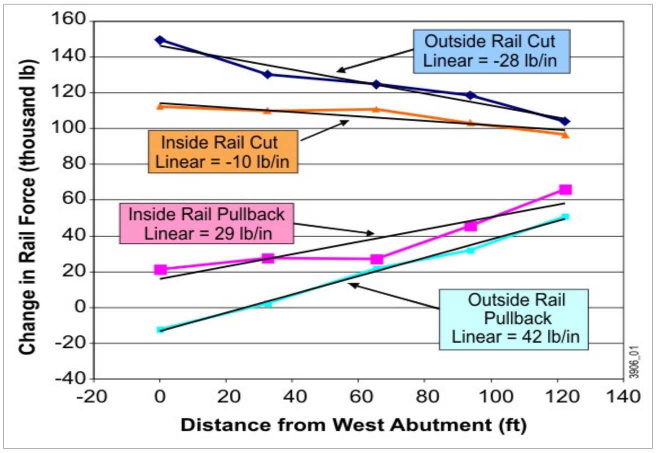

Figure 3. Rail Force Results

Figure 4 displays measured deflections both for rail cut and pull back scenarios on the inside rail. Results for the outside rail were similar. Girder displacements were negligible. A maximum displacement of about 0.75 inch was seen at the rail cut. Note that the displacement is variable along the span and that the tie-to-girder displacement is generally greater than the rail-to-tie displacement. This indicates that the smooth top girder often has lower longitudinal restraint than the box anchored rail-to-tie interface.

About 80 percent of the displacement was between the smooth top girder and tie. Only about 20 percent of the movement was between the rail and tie.

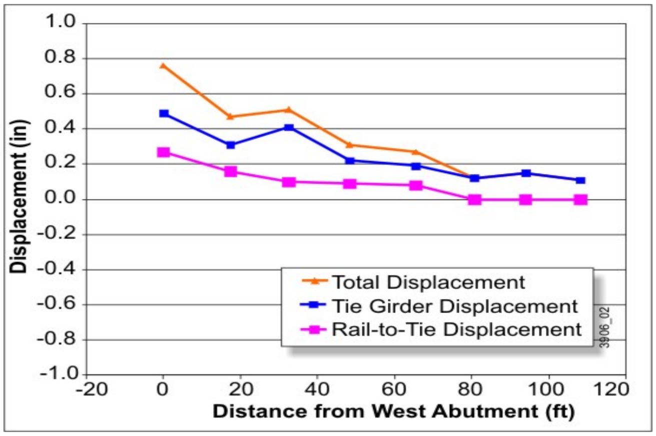

Figure 4. Displacements Due to Rail Cut on Inside Rail

Table 1 summarizes the measured values. Note that the longitudinal restraint due to a gradual application of force is on average 16.5 pounds per inch higher than the sudden application of force from the rail cut. Also note that the outside rail exhibited higher restraint than the inside rail which had all new fasteners. This suggests that for this bridge, as discussed below, the longitudinal restraint may be governed by the deck to girder interface.

Table 1. Measured Longitudinal Restraint (pounds/inch)

	Inside Rail	Outside Rail	Average
High Strain Rate	10	28	19
Gradual Strain Rate	29	42	35.5
Ratio	35%	67%	54%

The CWR model assumes a simplified rail-to-deck longitudinal resistance (combination of rail-to-tie and tie-to-girder) diagram similar to the one shown in Figure 5. An initial linearly ramped resistance is followed by fully activated friction at a constant resistance in pounds per inch of rail. Based on the test results, constant resistance values of 19 pounds/inch for high strain rate applications and 36 pounds/inch for gradual strain rate applications may be appropriate. Insufficient data exists to determine the slope of the initial ramp; however, it is assumed to be fairly steep, with full friction being activated within 1/16 to 1/8 inch of displacement.

Rail anchor testing was based on AREMA guidelines followed by additional force application. A total of 12 anchors were tested. Figure 6 shows an example of typical results. The results indicate that under gradual strain rate application, rail anchors may be capable of significantly higher restraint than were observed on the bridge. Higher restraint could be at least partially realized with a higher restraint deck-to-girder fastening system.

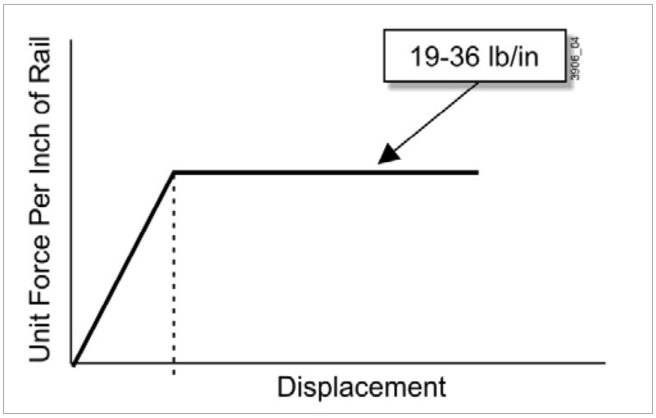

Figure 5. Simplified Representation of Bridge Longitudinal Restraint

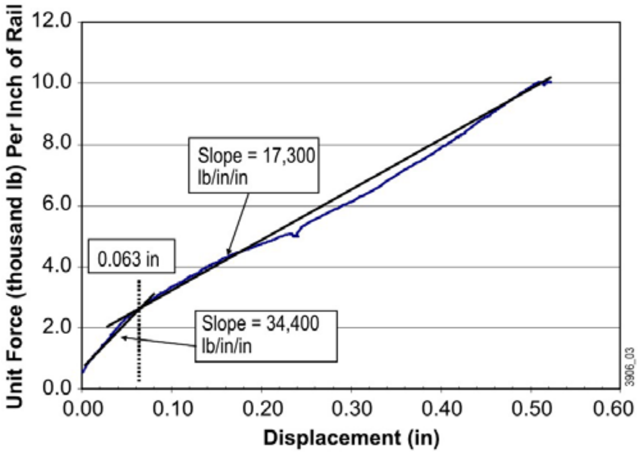

Figure 6. Example of Rail Anchor Test Results

FUTURE WORK

TTCI is in the process of testing a 400-foot open-deck steel bridge with a variety of CWR anchoring conditions. This testing will complete calibration and validation of the model. The model should be capable of assisting bridge engineers in determining the best CWR policies for particular bridges.

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