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Impact in Short Steel Ballast Deck Bridge Spans

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Summary

Recent bridge testing and research by Transportation Technology Center, Inc. (TTCI) could lead to significant savings by delaying the replacement of steel railroad bridges. An extended life can result from a better estimate of fatigue life for steel bridge components, particularly the shorter members that typically are the most prone to fatigue.

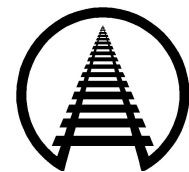
TTCI engineers tested four ballast deck steel bridge spans on the Union Pacific Railroad to measure impacts under train traffic at various speeds. The study's purpose was to develop and recommend appropriate values of impact to be used for fatigue assessment of steel bridges. Preliminary findings are:

- Use of more appropriate impact percentages for fatigue assessment can result in an increase of more than 100 percent in estimated fatigue life for steel bridge components.
- For fatigue ratings, the use of a reduced impact can show a capacity increase sufficient for heavy axle load (110 tons or more) traffic instead of 100-ton traffic in some cases.
- Short steel bridge members influenced by multiple wheel loads, such as floor systems, which are the most sensitive to fatigue, would realize the greatest increase in estimated fatigue life.
- Slid-flat wheels and low rail joints produced the highest impact readings. A few measured impacts exceeded the design impacts.
- Reducing the number of high impact wheels, rail joints, and overloaded cars can reduce the stress state of railroad bridges, resulting in extended service life and reduced maintenance.
- Reducing train speed over a bridge can reduce the forces imparted to the bridge until other appropriate actions can be taken.
- For purposes of fatigue rating, 35 percent of the current design impact could be used for beams, stringers, and floor beams of steel bridges for spans up to 50 feet long.

Better assessment of bridge capacity can lead to better prioritization of bridge maintenance and capital expenditures. In some cases, a better fatigue assessment might allow a railroad to postpone a costly bridge upgrade or replacement.

Results of this investigation are being shared with the American Railway Engineering and Maintenance-of-Way Association (AREMA) Committee 15 – Steel Structures, in order to affect changes in the guidelines for capacity assessment of steel bridges.

This work was performed as part of the AAR's Strategic Research Initiatives Program on steel bridge life extension.



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Technology Center, Inc.

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INTRODUCTION AND CONCLUSIONS

Recent bridge testing by TTCI could lead to significant savings by delaying the replacement of steel railroad bridges. An extended life can result from a better estimate of fatigue life for steel bridge components, particularly the shorter members that typically are the most prone to fatigue. Preliminary calculations show that the use of a more appropriate impact value for fatigue assessment can result in an increase of more than 100 percent in estimated fatigue life for some steel bridge components. Better assessment of bridge capacity can lead to better prioritization of bridge maintenance and capital expenditures. This is especially important as railroads upgrade bridges to carry heavy axle load (HAL) traffic. In some cases, a better fatigue assessment might allow a railroad to postpone a costly bridge upgrade or replacement.

Impact loads on railway bridges are the result of wheels with high impacts, overloaded cars, rail joints, and vehicle dynamics. Minimizing these items can reduce the stress state in bridges.

The impact values used for design of steel railway bridges, as found in the American Railway Engineering and Maintenance-of-Way Association (AREMA) manual, have a very low probability of occurrence. This level of impact is likely to happen only a few times in the life of a bridge. If these impact values are used to rate a bridge for fatigue, the fatigue life of many bridge members is underestimated. This is particularly true for span lengths of 30 feet or less that have high design impact factors and experience high numbers of load cycles. The reason is that the high stress levels using the design impact are assumed to occur during every load cycle, rather than infrequently. The result is a shorter estimated fatigue life.

The purpose of this study is to determine appropriate impact values for capacity rating and fatigue evaluation. The impact values should reflect current equipment and bridge maintenance practices. The AREMA design impacts are based on tests of several bridges during the 1960's and earlier. Since then, wheel loads, suspension systems of railcars, wheel maintenance, and rail maintenance practices have changed, resulting in a need to re-evaluate impact on bridges.

Research Program

TTCI researchers gathered steel bridge impact data from member railroads and previous AAR field tests conducted on numerous steel spans. The data was

primarily from open deck spans. No data was available for ballast deck spans 30 feet long or shorter.

Researchers also conducted tests on four spans of three ballast deck steel bridges on the Union Pacific (UP) Railroad. The purpose of these field tests was to obtain impact data for ballast deck steel bridge components with spans less than 30 feet long. The field tests supplemented the previous data for the purpose of determining a suitable impact to be used for fatigue rating of short steel spans.

Several stringers, floor beams, and main girders on the UP bridges were instrumented with strain gages. The strain gages were mounted at mid spans on the bottom flanges of stringers and floor beams. In addition, vertical force circuits measured wheel loads on both rails.

Over each bridge, a test train passed first at 2 mph and then at 5 mph to 40 mph (maximum authorized track speed) in 5-mph increments in both directions. Test trains had one or two 4-axle locomotives at each end. Test trains each had 10 loaded cars with gross rail loads from 124 tons to 148 tons, with most cars weighing about 131 tons. Only one car of 30 was above the 143-ton load limit for the line.

TEST RESULTS

The measured data was used to determine the statistical distribution of impacts, as well as to investigate the effects of span length and train speed. Impact was calculated as the ratio of peak strain at a particular train speed to the corresponding peak strain for the 2 mph run in the same direction. Impact was calculated for each peak of a load cycle during a train pass. Most members experienced about one load cycle per group of closely spaced axles of the test train. So each train pass generated about 15 distinct load cycles per train for most members for the test trains used.

Figure 1 shows the measured impact data plotted against the span lengths for different members. The trend indicates that impact reduces with increase in length. This is expected. Longer spans support more axles, so the effect of impact from a single axle is reduced. This trend is consistent with the AREMA design guideline for impact. Note that most of the measured impacts are significantly less than the design guideline.

Figure 2 shows the measured impact data plotted against the speed of the test train for all span lengths tested. The trend indicates that impact increases with increase in train speed. Again, this is

expected to an extent. For impacts due to flat wheels and rail surface anomalies, higher speeds lead to higher impact. But for vehicle dynamic behavior, there is often a resonant speed that produces the highest impact. For example, wheel loads due to car rocking might be higher at 20 mph than at 40 mph. The trend in the data is generally consistent with the AREMA guideline for impact as a function of speed. This data supports the practice of using slow orders to reduce forces imparted to bridges.

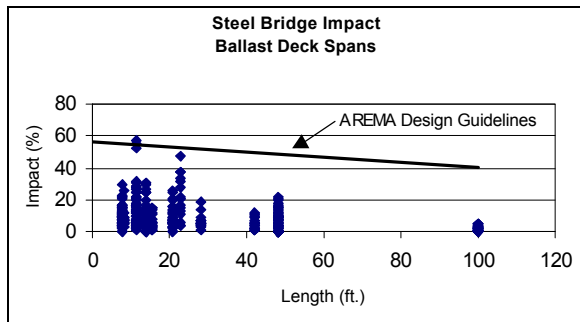


Figure 1. Effect of Span Length on Impact

To determine an appropriate amount of impact to use for fatigue rating purposes, the measured impacts were compared to the calculated design impacts for the same member. Figure 3 shows a normal probability plot of the distribution for the ratio of the measured to calculated design impact for span lengths up to 50 feet. This is based on measurements of 23 members. Calculated design impacts include the ballast deck adjustment factor and appropriate rocking effect for each member.

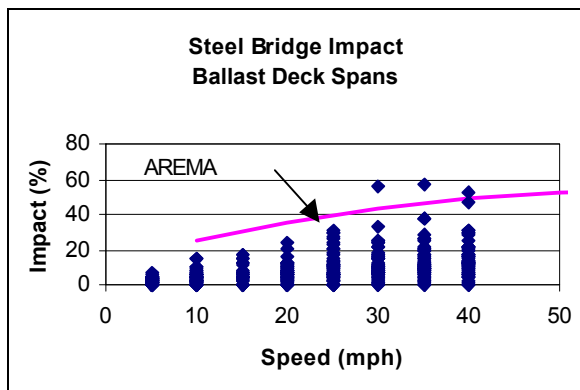


Figure 2. Effect of Train Speed on Impact

From Figure 3, the 95th percentile for the ratio of measured impact to calculated design impact is 31 percent. In other words, only 5 percent of the measured impacts exceed 31 percent of the design impact. The 95th percentile is consistent with the statistical probability used in the development of the

fatigue criteria used in the rating of steel bridges. Similar results were obtained using collected data from previous member railroad and AAR tests on open-deck steel bridge bending members including stringers, floor beams, and girders. These results suggest that for fatigue capacity rating of beams, stringers, and floor beams of steel bridge spans, an impact equal to 35 percent of the current design impact percentages is appropriate.

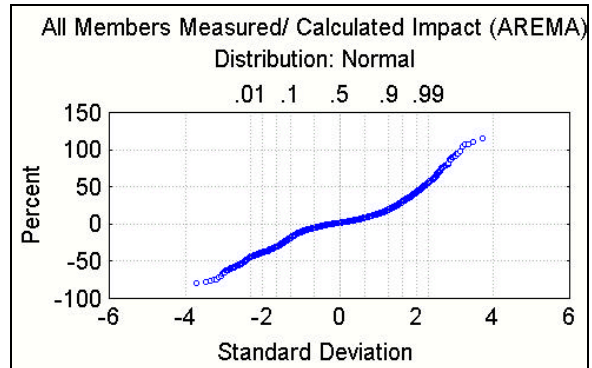


Figure 3. Statistical Distribution of Measured/Calculated Design Impact

For example, take a steel bridge component with a design impact of 48 percent. For fatigue rating purposes, 35 percent of 48 percent equals 17 percent. So the stress range used in the fatigue rating would be 1.17 times the static stress range, rather than 1.48 times the static stress range. The difference, 1.48/1.17 is a factor of 1.26. This is greater than the increase in stress range caused by 315-kip HAL cars instead of 263-kip cars. Since fatigue life varies with the cube of the stress range, the fatigue life using the reduced impact is improved by a factor of $(1.26)^3$, or 2.0. The amount of impact varies with member length and other factors, so the actual benefits will likewise vary.

Note also from Figure 3 that only a small percentage of measured impacts exceed 100 percent of the calculated design impact. This indicates that the current design impact values seem to be appropriate for design of steel bridges. The data does not suggest that the design impact values need revision.

Preliminary calculations for steel bridge members with 30-foot spans indicate that the estimated fatigue life could be at least 100 percent higher, using 35 percent of the design impact as compared to the full-design impact. The estimated fatigue life increase could be even greater if the stress level with the reduced impact is below the endurance limit stress.

Note that the data collected reflects impacts due to factors including vehicle dynamics, wheel flats, and rail anomalies. But it does not include the effects of overloaded cars.

According to train lists furnished by the train crew, only 1 car of the 30 used in these tests weighed more than the 143-ton load limit for the line. Reducing the incidence of overloaded cars can further reduce the stress state on railroad bridges.

Bridge Descriptions

To supplement the data from previous member railroad and AAR bridge tests, TTCI tested four spans of three bridges located on the UP's Tennessee Pass Subdivision. These bridges all have ballast decks and are located near Adobe, Florence, and Cañon City, Colorado, on single-track territory.

The bridge at Adobe is about 146 feet long with three through-plate girder (TPG) spans. Each span is 48 feet long. The floor systems have four rows of stringers. The first span has 7-foot, 9-inch long stringers and the other two spans have 11-foot, 6-inch long stringers. The floor beams are 16 feet long. The bridge has 115-lb/yd jointed rail. TTCI tested the first and third spans of this bridge (Figure 4).



Figure 4. Three-Span TPG Test Bridge on UP at Adobe, Colorado

The bridge at Florence is a 102-foot long TPG span. Its floor system has two rows of stringers. The stringers are 14 feet long. The bridge has 136-lb/yd continuous welded rail (Figure 5).

The bridge at Cañon City is a 125-foot long through truss. Its floor system has two rows of

stringers. The stringers are 21 feet long. The bridge has 115-lb/yd continuous welded rail (Figure 6).



Figure 5. Single-Span TPG Test Bridge on UP at Florence, Colorado



Figure 6. Single-Span Through Truss Test Bridge on UP at Cañon City, Colorado

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