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## The Effects of Moveable Bridge Joint on the Fatigue Life of Welded Braces of Open Deck Steel Bridge at FAST

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### Summary

Under the Association of American Railroads' Strategic Research Initiatives Program, Transportation Technology Center, Inc. is studying techniques to reduce the stress-state of steel bridges at the Facility for Accelerated Service Testing (FAST).

A moveable bridge rail joint particularly increased the rate of crack initiation and propagation in the steel bridge braces at FAST. The current test was conducted after the joint was replaced with continuous welded rail. This test has led to the following conclusions:

- In both the tests, with and without the joint, the measured stresses in some members exceeded the fatigue design criteria of current American Railway Engineering and Maintenance of Way Association (AREMA) guidelines.
- After removing the joint, the stresses in some members were reduced by up to 20 percent. As per AREMA fatigue life rating guidelines, this reduction in stress tend to double the fatigue life of steel braces.
- The impacts were particularly higher in the bracing members directly under or close to the location of the joint.
- The design of the bracing members in both the spans of the steel bridge at FAST, with and without the joint, meet the current strength and stability criteria of AREMA guidelines.
- Modeling of axial and bending stresses in lateral braces shows that the stress-state may be reduced by using symmetric sections as bracing members.
- Use of bolted bracing members is recommended. Such members have higher fatigue strength over welded ones. With bolted bracing members, fewer bending stresses can be induced. In welded members, bending stresses tend to be highest near the welds, increasing local stresses.

As a result of the current tests, the fatigue design and rating of steel bridge bracing members under heavy axle load traffic are expected to be reviewed. Also, economical and more reliable bracing repair techniques are expected to be developed.

These findings will be used to extend the service life of existing rail bridges. Results will be shared with AREMA committees to improve bridge designs.



## INTRODUCTION AND CONCLUSIONS

Steel bridges on some heavy axle load (HAL) routes are being subjected to unprecedented levels of traffic. In less than 2 years, some bridges encounter more than 2-million cycles of fatigue, which is the current criterion of the fatigue design of some bridge components. Demands for increasing speeds and axle loads have particularly increased the maintenance requirements of steel bridge braces. Today, the repair of braces makes up a major portion of the steel bridge maintenance budget. This load environment has prompted a review of the current design criteria for steel bridge braces subjected to HAL traffic.

Steel bridge braces are designed for axial loads imparted to the bridge primarily from lateral forces of equipment and wind loads. The current codes of practice consider only direct axial stresses for the design of lateral braces. Rising train speeds and car loads may cause considerable impacts, rotation of flanges, unequal deflection, and out-of-plane bending. All of these factors may create significant bending and torsional stresses in braces in addition to direct axial stresses.

In particular, rail joints over bridges increase the impacts from wheels. These impacts can reduce the fatigue life of bridge members substantially, thus, increasing the repair and maintenance requirements of bridge members.

A moveable bridge rail joint was installed on the steel bridge at FAST for endurance testing. During the life of this joint, the stresses in the lateral braces were measured and results documented in *Technology Digest* TD-06-013.<sup>1</sup> Later, the joint was removed. Continuous welded rail was installed on the bridge and the stresses in braces were measured again. This TD discusses and compares the level of axial and bending stresses in bracing members with and without the joint.

In general, the stresses in members welded to compression flanges decreased after the joint was removed. The effect was particularly significant in members closest to the joint, where up to 20 percent reduction in stresses was measured.

The maximum axial stresses in the current test were within the constant amplitude fatigue limit for member connections of this type. The total of measured axial and bending stresses in some members, welded to compression flanges, exceeded the constant amplitude fatigue limit. There was no measurable effect measured in the stress-state of members welded to bottom of girders.

Finite element modeling results show that the asymmetric sections of braces, mainly due to out-of-plane bending and shear lag, may create very high local stresses in the welds, and that these stresses may be reduced by using symmetrical cross-sections.

## BRIDGE DESCRIPTION

The steel bridge at FAST has two deck plate girder spans: 65- and 55.5-foot spans. All of the members are welded. The braces are angles or T-Sections. Except for the horizontal braces that have both legs welded, only one leg of all other braces is welded at the ends. Top and lateral braces are

welded to every other opposite stiffener. The bottom lateral braces are connected to gusset plates, which are welded to the web. Top lateral braces are directly welded to the flanges. The 65-foot span was designed in 1956 for a Cooper E-72 loading with diesel impact. For comparison, the HAL train is equivalent to about an E-56 loading on the main member of this span, although for individual components, this can be higher due to the 79,000-pound axle loads of the test train at FAST. The diaphragms are at 16-foot spacing. All bracing members are 4- by 4- by 1/2-inch angles.

The 55.5-foot span was designed in 1967 for a Cooper E-72 loading with diesel impact. The HAL train is equivalent to about an E-61 for the main members of this span, although for individual components, this can be higher due to the 79,000-pound axle loads of the test train at FAST. The diaphragms are at 11-foot spacing and all the diaphragm braces are of L-Section 5 by 5 by 1/2 inch. Top and lateral braces are ST 6 by 20.4. The moveable bridge joint is at the center of bridge with parts on each span.

## METHODOLOGY

Strain gages were applied at the end and midspan diaphragms at selected members, as Figure 1 shows at the following locations:

- Top and bottom lateral braces: X and Y bending at 1/4 and 1/2 spans and axial forces at 1/2 span
- Top lateral braces (at mid diaphragm only): X and Y bending at 1 inch from welds
- Horizontal diaphragm members: X and Y bending at 1/4 and 1/2 spans and axial force at 1/2 span
- Cross diaphragm members: X and Y bending at 6 inches from connection and axial force at 1/4 span



Figure 1. Strain Gaged Members of Bridge Spans

All of the strain gage circuits were full Wheatstone bridges with compensation for thermal changes. The axial strain gages were installed on the neutral axes of the sections. The bending circuits were installed at 0.75 inch from the edges.

First test runs were at 2 miles per hour (mph) in each direction. The speed was then increased to 5 mph with 5 mph increments up to 45 mph. The test train had two 4-axle locomotives and fourteen 315,000-pound cars.

**STRESS-STATE OF BRACING MEMBERS**

Ninety ninth and 95<sup>th</sup> percentile impacts in all members were calculated using 2 mph as a reference and grouped as mid and end diaphragms. A histogram for each diaphragm was then plotted showing the impact frequency. As Figure 2 shows, the 99<sup>th</sup> percentile impacts considerably reduced in end diaphragm members of the 55.5-foot span after the joint was removed. The diaphragm was 6-feet away and was closest to the joint. Figure 3 estimates up to a 20 percent reduction in 99<sup>th</sup> percentile impacts. Insignificant reduction in impacts was observed in other diaphragms, which are 25 to 60 feet away from the joint. Figure 4 shows that the results of 95<sup>th</sup> percentile were consistent with the 99<sup>th</sup> percentile.

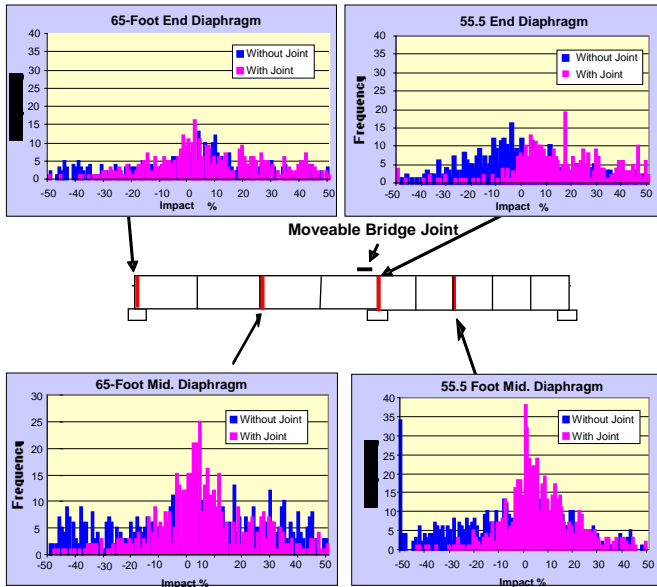


Figure 2. Impacts-Frequency in Diaphragms, 99<sup>th</sup> Percentile

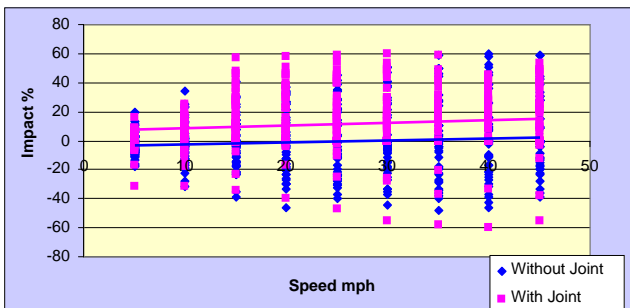


Figure 3. Reduction in Stress after the Joint was Removed on the 55.5-foot End Diaphragm

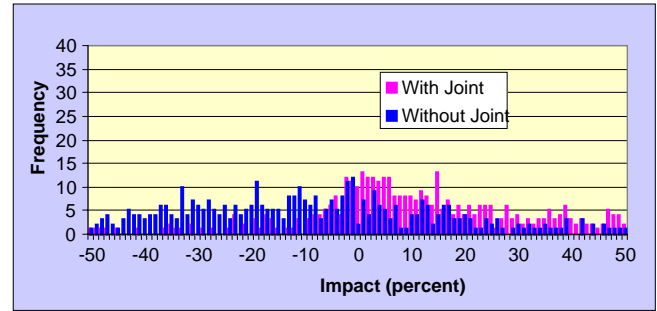


Figure 4. 55.5-foot End Diaphragm, 95<sup>th</sup> Percentile

Figure 5 shows maximum stresses in all diaphragms. There was no significant change in both with-joint and without-joint stresses. The maximum axial stresses were well within the fatigue design limit. But total stresses, sum of axial and bending, in some members exceeded the design stress. This conclusion suggests that bending stresses should be considered in the design of lateral braces.

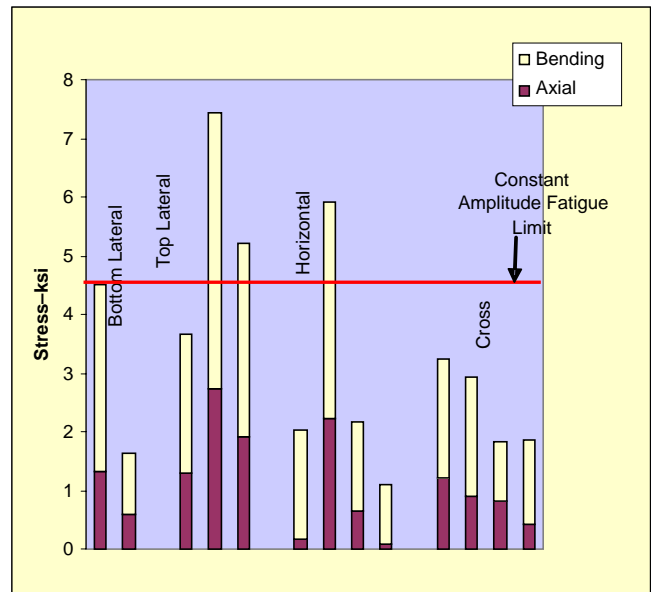
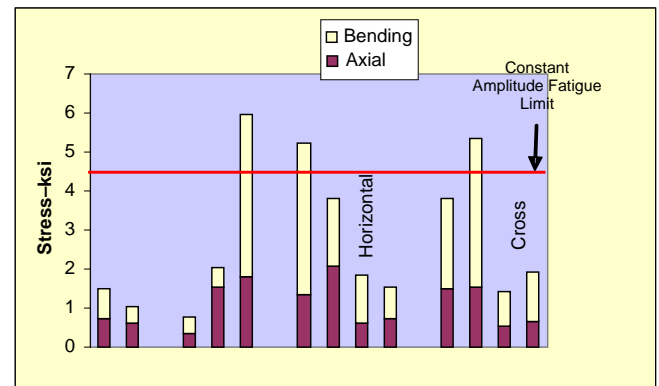
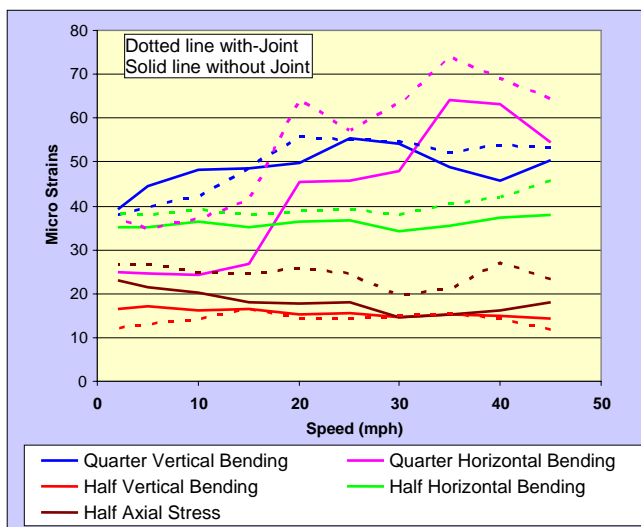


Figure 5. Maximum Stresses for 55.5- and 65-foot Diaphragms

In general, stresses in horizontal diaphragm members and top lateral bracing members were higher than bottom lateral members and cross diaphragm members. This is probably because the members are connected to compression flanges and are subjected additional stresses due to flange rotation and out-of-plane bending of girders.

The 99<sup>th</sup> percentile of total stress was calculated for each member. Figure 6 compares the results of bending and axial stress, with-joint and without-joint for one member. In general, the magnitudes of bending stresses were higher than the axial stresses. Also, the effect of a moveable bridge joint was more significant on bending stresses than the axial. In addition, vertical bending and axial strains increased with the speed. The effect of speed was negligible on horizontal bending stresses.



**Figure 6. 55.5-foot Span End Diaphragm Bending and Axial Strains in a Typical Bracing Member, 99<sup>th</sup> Percentile, Top Horizontal Member**

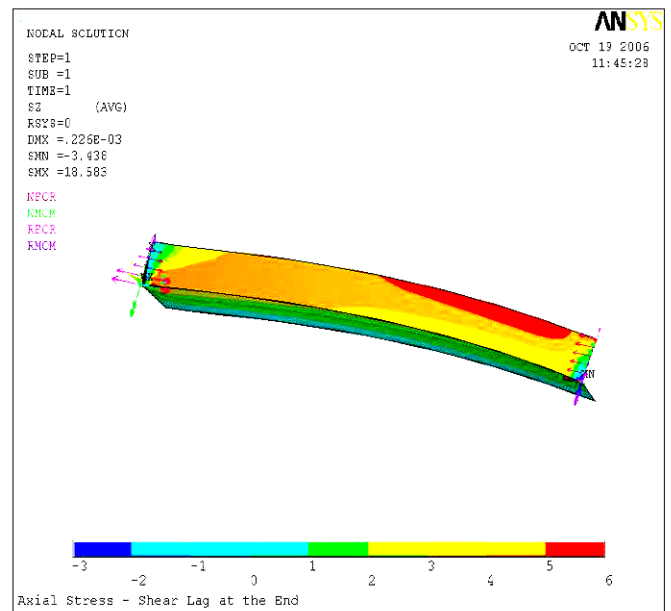
The measured stresses were higher in the vertical leg of L-Section than that of the horizontal leg at mid and quarter span of the members. However, at 2 inches from the end, the situation was opposite. This is understandable since only the horizontal leg of the member was welded to the compression flanges. The stresses from both the legs can only be transferred through the horizontal leg at the end, causing stress concentration. The axle loads of the 315,000-pound cars at FAST are similar to those of the Cooper E-80 design load. For 286,000-pound cars, the stresses would be expected to be about 10 percent lower than those measured and presented herein.

**FINITE ELEMENT ANALYSIS**

A simple model of a lateral brace was analyzed to study the effects of asymmetric section at the member ends. An L-Section was meshed with shell elements. One leg at one end of the members was fixed in all directions. A force was applied at the far-end parallel to the longitudinal direction of

the member. The magnitude of force was equal to the typical measured value at FAST.

The uniform axial load resulted in highly non-uniform reaction at one end, as Figure 7 shows. This high-stress concentration appears to be due to out-of-plane bending and shear lag. Also, the local stiffness is higher at the intersection of two legs, and therefore tends to attract more stress. The model was also analyzed with both bending and axial force at the same time. The first principal stress and the longitudinal stress were almost identical. This shows that torsional stresses due to out of bending are negligible.



**Figure 7. Stress Plot of Lateral Brace Subject to Uniform Axial Load**

Use of bolted bracing members is recommended. Such members have higher fatigue strength over welded ones. With bolted bracing members, fewer bending stresses can be induced. In welded members, bending stresses tend to be highest near the welds, increasing local stresses.

**FUTURE WORK**

These findings will be used to extend the service life of existing rail bridges. Results will be shared with AREMA committees to improve bridge designs.

**REFERENCES**

1. Muhammad Akhtar, Duane Otter, and Brian Doe. May 2006. "Stress State Evaluation of Lateral Braces in Open Deck Steel Girder Bridge at FAST." *Technology Digest* TD-06-013. Association of American Railroads, Transportation Technology Center, Inc., Pueblo, Colorado.