Crack Development and Growth in the FAST Steel Bridge

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ABSTRACT

New cracks continue to develop, old cracks and crack repairs are being monitored, and various Non-Destructive Evaluation (NDE) techniques are being employed as test train operations continue on a two-span, welded deck plate girder railroad test bridge at the Transportation Technology Center near Pueblo, Colorado.

Since installation the bridge has accumulated over 6.4 million load cycles of heavy railroad traffic, including over 5.5 million load cycles on a significant crack in a tension flange. Cracks have developed and propagated in both the 65- and 55.5-foot spans of the steel bridge. Crack locations include web stiffener details, diaphragms, and lateral bracing.

Results from this ongoing fatigue test are intended to help railroads extend the safe service life of their steel bridges. As well, the monitoring tools developed and evaluated in this test will be valuable in helping railroads to better prioritize bridge maintenance and renewal budgets.

Keywords: welded girder, railroad, railway, crack growth, fatigue, steel bridge

1.0 INTRODUCTION

The Facility for Accelerated Service Testing (FAST) has been used for full scale railroad testing since 1976 at the Transportation Technology Center (TTC) near Pueblo, Colorado. The FAST track features a 2.7-mile loop with a wide variety of rails, ties, and other track components
subjected to railroad loading. A test train circles the loop approximately 500 times per week with an 80-car train. Car loading is 315,000 pounds gross rail load, which is about 10 percent higher than the current maximum loading for most North American rail lines.

In 1997, a two-span steel bridge was added to the test loop. See Figure 1. The test bridge has welded deck plate girder spans, with an open deck. The bridge was originally installed for purposes of testing bridge decks, rail anchoring and fastening systems for bridges, and bridge approaches. The original test plan did not call for testing of crack growth in the spans. But, due to rapid development of several cracks, the bridge soon became a full-scale fatigue and crack growth test while continuing to serve its intended purposes as well.

Figure 1: Two-Span Welded Steel Girder Railway Bridge at FAST.
The ongoing full-scale steel bridge fatigue and crack growth test at FAST offers a unique opportunity to monitor and evaluate crack initiation and growth in a controlled railroad environment. The bridge has now accumulated more than 6 million load cycles. With at least 35 cracks, it serves as an ideal test bed for development of various NDE techniques. And it also serves to test field repairs and retrofits.

2.0 GIRDERS DESCRIPTION

The welded deck-plate-girders installed on the steel bridge at FAST were designed in the 1950s and 1960s, according to the American Railway Engineering Association (AREA) and Pennsylvania Railroad practices at that time. The 55.5-and 65-foot span girders were fabricated in 1968 and 1957, respectively. The spans were fabricated with welding practices that were then considered acceptable, but have since been significantly improved. The weld techniques and details appear to be a contributing cause to crack initiation in the bridge.

The second-hand girder spans were donated by Conrail. The girders were designed for a Cooper E-72 loading with diesel impact. Prior to installation at FAST, they carried minimal traffic, mainly cars weighing 220,000 lbs or less. No cracks were visible at the time of installation. At FAST, the girders have been subjected almost exclusively to cars weighing 315,000 lbs. The current loading on the bridge is equivalent to about Cooper E-60 in flexure on the 55.5-foot span, and Cooper E-56 in flexure on the 65-foot span.

Both spans have two girders spaced at 6.5-foot centers. Both spans are braced with top and bottom lateral bracing and X-type diaphragms. Diaphragms are L sections in both the spans. Lateral braces are L sections in the 65-foot span and T sections in the 55.5-foot span.
3.0 Overview of Crack Development

During the initial 1.4-million load cycles of test train traffic over the bridge, many cracks developed in the tension regions, primarily in the older longer 65-foot span. Many of those cracks now appear to be dormant. The primary cause of the cracks seems to be in the stiffener-to-web weld details, which are not consistent with the current American Railway Engineering and Maintenance of Way Association (AREMA) guidelines [1]. Similar cracks have been experienced in both railway [2] and highway spans [3]. No new cracks were observed between 1.4 million and 3.5 million load cycles in the longer span. However, propagation and growth of existing cracks were monitored during this period.

Several new cracks developed after 3.5 million load cycles. Most of these more recent cracks are in the lateral braces and diaphragms, near the top of the spans; again, most are in the longer span. Nearly all of the new cracks were in the vicinity of bolted rail joints. It is likely that the impacts from train traffic over the discontinuous rail surface contributed to the initiation of these cracks. Other factors that may have contributed to initiation of these cracks include weld issues and out-of-plane bending. Some of the new cracks have been repaired with bolted splices. Other cracks are in areas not critical to the capacity of the bridge.

Strain gage measurements have been taken on various bracing and diaphragm members. The strains indicate that the axial forces are well within design limits. However, there is also significant bending in the various bracing members. When the axial and bending stresses are combined, they are higher than the fatigue stress recommended for the welded connection details in some members. Measured strains tended to be higher in the top lateral braces and diaphragm members as compared to the bottom members. This corresponds well with the cracking
experienced to date. Note that neither bending nor fatigue are typically considered in the design of bracing members for railway bridges.

Figures 2 and 3 show the initiation history of cracks during the service life of the girder spans at FAST. The higher number of cracks in the longer span might be attributed to the relatively lower lateral stiffness of the span as well as the different weld details and practices. The X-type diaphragms are spaced at 16 feet in the 65-foot span as compared to 11 feet in the 55.5-foot span. The lower lateral stiffness of the longer span due to diaphragm spacing may have allowed more out-of-plane bending that contributed to initiation of cracks in the regions normally considered as within the compression zone.

![Graph showing crack initiation history in 65-foot welded girder span](image_url)

Figure 2: Crack Initiation History in 65-foot Welded Girder Span
Figure 3: Crack Initiation History in 55.5-foot Welded Girder Span

The different types of cracks observed are:

- Cracks below intermediate web stiffeners (Figure 4)
- Crack in tension flange (Figure 5)
- Cracks in diaphragm members or connections (Figure 6)
- Cracks in lateral braces (Figure 7)
- Crack in the weld between stiffener and compression flange (Figure 8).
Figure 4: Crack in Web Stiffener Detail with Drilled Holes to Arrest Cracks

Figure 5: Crack in Tension Flange near Web Stiffener Detail
Figure 6: Crack in Diaphragm Connection

Figure 7: Crack in Lateral Bracing (top) with Bolted Splice Repair (bottom)
Transportation Technology Center, Inc. staff continues detailed bridge inspections about every 200,000 cycles. Thus far, there has been no experience to suggest that inspections are needed more frequently.

4.0 Crack Types Observed

4.1 Cracks below Intermediate Web Stiffeners

Cracks below the intermediate web stiffeners were the first to initiate, with several of the same type appearing shortly after the beginning of test train operations over the bridge at FAST. Cracks of this type have been reported in both highway and railway bridges with similar stiffener details [2,3]. On the insides of the girders, some of the stiffeners also serve as transverse connection plates. However, cracking has been observed at stiffeners both with and without
connecting members. Design guidelines for termination of welded stiffeners in the tension zone have since been changed to improve this detail [1]. All cracks of this type appear to have initiated in the welds between stiffeners and tension flange. Most appeared during early periods of service prior to 1.4 million load cycles. Nearly half (10 of 18) of these cracks have propagated into the girder webs. The others remain in only the welds at the time of this writing. Many of these cracks appear to have since become dormant. It is surmised that the cracking has relieved some of the local stress concentration initially associated with the weld detail, or the crack has propagated to an area where the local stresses are no longer great enough to drive crack growth. Overlapping welds may create complex residual stress distributions that may precipitate crack initiation.

Figure 4 shows a typical crack at the bottom of the stiffener, which was observed very early in the testing at 160,000 cycles. This particular crack has been arrested with drilled holes. To date, cracks of this type have been observed on 12 of 26 intermediate stiffeners on the longer span and 6 of 22 intermediate stiffeners on shorter span. The clearance between web-flange and web-stiffener welds in the longer span is 3/4 inch, and the welds overlap at the bottom of intermediate stiffeners. The clearance between stiffener and flange on the shorter span is 1 inch and the welds do not overlap. Presumably, the shorter span has fewer cracks because its web-flange and web-stiffener welds have more clearance than those in the longer girders, and the welds do not overlap.

Current AREMA recommended practice calls for a clearance of six times the web thickness, between the intermediate stiffener welds and the web-to-tension-flange welds. This is a distance of 3.0 inches in the shorter span and 3.75 inches in the longer span.
Many cracks that initiated in the welds have propagated into the webs. The longer and shorter spans experienced six and four such cracks, respectively. In the longer span, one crack was arrested with two 7/8-inch diameter holes shortly after it was discovered (Figure 4). No growth has been noted since. Another crack was arrested in the web with a drilled hole, but the other end of the crack propagated down into the tension flange within the following 300,000 load cycles (Figure 5). Only minimal crack growth has been noted since then. The other three web cracks in the longer span have not been treated, and appear to be dormant.

Two web cracks in the shorter span remain active. These cracks are under the first intermediate stiffeners of each girder near the bearings at the center pier. This location is below the area where moveable bridge miter rail joints were installed as part of another test program. Figure 9 shows that one of these cracks became more active during the time the moveable bridge rail joint was installed. Presumably the stresses associated with higher dynamic impact forces are responsible for the increased crack growth rate. The length of cracks in the north and south girders increased from 10 to 15 inches and 2 to 5 inches, respectively. Crack growth activity on these cracks slowed after the removal of the joint.
Figure 9: Crack Growth during Time Rail Joint was installed above Crack Location

4.2 Crack in Tension Flange

Figure 4 shows the web stiffener detail crack that propagated into the tension flange. The crack in the web was first noted, and the hole at the top of the crack drilled at about 230,000 cycles. At 550,000 cycles, the growth into the tension flange was noted. At that time, a support crib was constructed beneath the span, as can be seen in Figure 1. At about 910,000 cycles, shims were removed from the support crib to allow full deflection of the span. The intention is to monitor growth of this crack under train operations. The support crib remains in place without shims as a safety measure should a sudden failure occur. Several other safety measures, including deflection and strain measurements, and an alarm system are in place in order to continue safe test train operations.
Ultrasonic measurements show the crack area to be about 3 percent of the tension flange. Crack depth near the web is about 1 inch. The tension flange crack is checked weekly (about 30,000 cycles) using ultrasonic testing. As of this writing, about 5.9 million cycles have been accumulated over the cracked girder since the supporting shims were removed to allow full deflection. To date, no measurable growth has been detected using ultrasonic measurements. Also, negligible crack growth activity has been detected using acoustic emission equipment. Strain gage measurements indicate maximum nominal tensile stresses of 7 to 8 ksi in the tension flange under normal train operations, with an equivalent constant amplitude stress range of about 5.8 ksi from rainflow cycle counting. Fracture mechanics modeling of the crack indicates that it should continue to grow, but the growth rate might be too low to measure for some time. Nonetheless, this crack is monitored frequently.

4.3 Cracks in Diaphragm Members

On the north girder of the longer span, all angles of the diaphragms are welded to one side of the intermediate stiffeners at some locations. When the bridge was subjected to lateral loads or unequally distributed vertical loads, out-of-plane bending may have caused an eccentric resultant force on the stiffeners. Three out of five diaphragms with these connection details initiated cracks that have propagated into the stiffeners (Figure 6). On the south girder, diaphragm angles tend to be welded on both sides of the stiffener at the top of the girder. The applied diaphragm forces are more balanced. No crack has been observed in the south girder diaphragm connections.

4.4 Cracks in Lateral Braces

To date, the longer span has experienced four cracked lateral braces between top flanges (Figure 7a). They have been repaired with bolted splices (Figure 7b). In one of these braces, the
crack was not completely through the cross section until after the splice was installed. It appears that these cracks initiated in welds, due to stress related to higher dynamic loads from rail joints in the track on the bridge. Each cracked brace is in the vicinity of locations where bolted rail joints were installed at one time. No cracks were observed on these braces before joint installation.

As noted above, strain gage measurements in both bracing and diaphragm members show the combined axial and bending stresses to be above the recommended fatigue stress for the welded connection details in some members. Normally, bending and fatigue are not considered in the design of these members.

4.5 Crack in Weld Detail of Stiffener and Compression Flange

Figure 8 shows a crack that recently initiated on the longer span. There is no diaphragm or lateral bracing attached to this stiffener. Rotation of the compression flange or out-of-plane bending of the web may have contributed to initiation of this crack.

5.0 TREATED WELD DETAILS

Some vertical stiffener welds in the shorter span were treated with Ultrasonic Impact Treatment (UIT). UIT is capable of reducing or reversing residual tensile stresses in weld and web areas by introducing compressive stresses. UIT also smoothes the weld profile, providing a better fatigue detail. The change in stress measured during application at TTC was about 8 ksi. See Figure 10. The stress change was measured using strain gages attached to the reverse side of the treated area. The gages were monitored throughout the treatment process. The thickness of the girder web is ½ inch.
Figure 10: Change in Strain during Application of Ultrasonic Impact Treatment to Left then Right Sides of Weld Detail.

6.0 EXPERIMENTAL WELD REPAIR OF CRACKED DIAPHRAGM

Figure 11 shows an experimental weld repair recently applied to portion of a web stiffener connected to diaphragm members. The crack began at the top, in the weld between the top horizontal member and the stiffener. It then progressed downward through the stiffener-to-web weld. Then it turned into the stiffener, and rapidly moved through the stiffener.
This is the first field weld repair attempted on this bridge. This repair is very unusual in that only the portions of the crack in the stiffener have been repaired. The portion of the stiffener between the repair welds is not connected to the web. The reasons for this unique repair are twofold: First, the intention is to attempt a repair weld to serve as a control for a later repair to be treated with UIT. By making only a partial repair, it is thought that cracking might happen sooner, thus shortening the test time required. Secondly, by not welding to the web, the chance of introducing a crack in the web is minimized. Such a crack might reduce the capacity of the bridge and preclude testing of a second weld repair with treatment.
7.0 **Effect of Joints in Rail Surface**

The rail surface over the bridge has changed significantly over time. For the first 3.1 million load cycles, the bridge had continuous welded rail, with no joints or rail surface discontinuities. This rail provided a smooth ride, with minimal impact. During the period from 3.1 million to 3.5 million cycles, a set of two-piece austenitic manganese steel (AMS) casting moveable bridge miter joints were installed on the bridge deck. This installation introduced significant impacts into the bridge due to the change in running surface. The rails from this moveable bridge miter joint were attached to the running rails using conventional bolted joint bars. The rail surface was discontinuous at five locations per rail during this test. The miter joint was located over the centre pier of the bridge.

At 3.5 million cycles, the AMS joint castings and assemblies were removed and replaced with bolted plug rails. The bolted plug rails were attached to the running rails using the same conventional joint bars, leaving two rail surface discontinuities per rail over the bridge. Those conventional joints remained in place until 4.9 million cycles.

From 4.9 million cycles to 6.2 million cycles, a set of premium quality moveable bridge joints was installed on the bridge. The joints were welded to the adjacent running rails. The only rail surface discontinuities were one per rail at the joint itself.

The installation of these test joints caused increased dynamic vertical and lateral loads, which may have induced cracking in some lateral bridge braces. The impact as measured using load-measuring wheels on a test train shows dynamic vertical wheel load increases of about 50% at typical train operating speed of 40 mph.
8.0 Fatigue Analysis

The fatigue life of each girder was estimated based on strain gage readings and steel bridge rating information as found in the AREMA guidelines. The estimated fatigue life was calculated for each tension flange of each span using measured strain data. Table 1 shows the Equivalent Constant Amplitude Stress Range (ESR), estimated fatigue life of tension flange, and deflection of each girder at typical test train speed.

Table 1: Girder Performance

<table>
<thead>
<tr>
<th>Girder</th>
<th>Equivalent Stress Range (ksi)</th>
<th>Fatigue Life (million cycles)</th>
<th>Typical Deflection under HAL train (inch)</th>
</tr>
</thead>
<tbody>
<tr>
<td>65-ft North</td>
<td>5.8</td>
<td>22</td>
<td>0.52</td>
</tr>
<tr>
<td>65-ft South</td>
<td>5.8</td>
<td>22</td>
<td>0.46</td>
</tr>
<tr>
<td>55.5-ft North</td>
<td>7.1</td>
<td>12</td>
<td>0.42</td>
</tr>
<tr>
<td>55.5-ft South</td>
<td>7.3</td>
<td>11</td>
<td>0.50</td>
</tr>
</tbody>
</table>

Based on these computations, the girders have several million cycles of remaining service life. The current loading on the bridge is equivalent to about Cooper E-60 in flexure on the shorter span, and Cooper E-56 in flexure on the longer span. This is lower than the Cooper E-72 loading for which the spans were designed. The measured stresses are also lower than current recommended design stresses. The ESR in each girder is lower than the constant amplitude fatigue limit stress for Category C details. These low stress levels may explain in part why the tension flange crack has not grown measurably for several million cycles.
The actual deflections are considerably less than the L/800 maximum recommended in the AREMA manual (0.83 inches and 0.98 inches for the shorter and longer spans respectively).

9.0 ACOUSTIC EMISSION TESTING

An acoustic emission system has been used to monitor activity of various cracks in both spans. The acoustic emission count activity generally correlates reasonably well with observed crack growth. But the observed crack growth rates have not been smooth. Instead the growth pattern has been more like a stair-step function, as shown in Figure 12. For this particular crack, a test duration of about 100,000 cycles was needed in order to determine the long-term average crack growth rate.

![Figure 12: Acoustic Emission Data from Growing Crack on FAST Bridge.](image)

10.0 SUMMARY AND CONCLUSIONS

With the accumulation of over 6 million load cycles of railroad traffic since 1997, the steel railroad bridge at the Facility for Accelerated Service Testing (FAST) has remained in service for a variety of ongoing full-scale fatigue tests. Various NDE techniques have also been
employed, both for evaluation of cracking, as well as for development of better procedures for use of NDE.

Cracks have developed and propagated in both spans of the steel bridge. Safety inspections every 200,000 load cycles and appropriate maintenance have kept the bridge safe for continued train operations. Crack initiation to date seems to all have occurred at welds. Thus far, the rate of crack initiation and crack growth has not indicated a need to inspect more frequently. Repair and remediation measures have thus far proven effective.

11.0 ACKNOWLEDGEMENTS

The authors are grateful for the support of the sponsoring organizations: the Association of American Railroads, and the U.S. Department of Transportation, Federal Railroad Administration. The authors are also thankful for the guidance and support provided by railroad bridge engineers, in particular Dr. Robert A.P. Sweeney, William G. Byers, and John F. Unsworth, past and current chairs of the AAR’s bridge research advisory group.

REFERENCES


LIST OF TABLES AND FIGURES:

Table 1: Girder Performance

Figure 1: Two-Span Welded Steel Girder Railway Bridge at FAST.
Figure 2: Crack Initiation History in 65-foot Welded Girder Span
Figure 3: Crack Initiation History in 55.5-foot Welded Girder Span
Figure 4: Crack in Web Stiffener Detail with Drilled Holes to Arrest Cracks
Figure 5: Crack in Tension Flange near Web Stiffener Detail
Figure 6: Crack in Diaphragm Connection
Figure 7: Crack in Lateral Bracing (top) with Bolted Splice Repair (bottom)
Figure 8: Crack in Weld between Stiffener and Compression Flange
Figure 9: Crack Growth during Time Rail Joint was installed above Crack Location
Figure 10: Change in Strain during Application of Ultrasonic Impact Treatment to Left then Right Sides of Weld Detail.
Figure 11: Experimental Weld Repair of Stiffener with Drilled Holes
Figure 12: Acoustic Emission Data from Growing Crack on FAST Bridge.