SHOULD WELDING BE USED TO REPAIR STRUCTURAL STEEL?

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INTRODUCTION

In the past, welding was used indiscriminately to repair railroad bridges, sometimes causing worse problems than the repair was intended to fix. Thus, in recent years there has been a fear of welded repairs, to the point that they are not being used in cases where they should be. The fears about welding are well founded. During the late 1960's and early 1970's many fatigue cracks developed in welded bridges. Most of these cracks can be attributed to a lack of understanding of fatigue and to the use of fatigue prone details. Therefore, cracking in these early welded bridges should not lead us to an out-of-hand dismissal of welded repairs.

This paper discusses common fatigue problems, what to consider when evaluating the use of a welded repair and then presents a case study where welding was successfully used to repair cracks in welded thru-plate and deck-plate girder spans.

REPAIRS

The most common reasons for repairs are corrosion, cracking, strengthening and retrofitting. This paper focuses on repairs of cracked members but many of the items discussed are applicable to any type of repair. Repairing a cracked member usually consists of either bolting on splice plates to redistribute the load, welding the crack or replacing the member in-kind. No matter what crack repair is selected, it is essential to determine the cause of the distress.

Repairing a crack without understanding and eliminating or mitigating the reason the crack occurred will usually result in the repaired member cracking again. If it took 50 years for the crack to develop and the span will be replaced in the next 10 years, repair of the crack alone may be acceptable. However, if this crack occurred, then chances are other cracks are likely to develop in similar details on the same span. By identifying and eliminating or mitigating the cause of the crack for all similar situations on the span, future cracking can be avoided.
FATIGUE

General

Although cracking can be caused by reduction of section from corrosion, most cracking is caused by fatigue. Fatigue is the process of the formation and propagation of a crack caused by repeated or fluctuating loading. The stresses caused by the repeated or fluctuating loading are additive to the existing stresses which may be residual.

All welding processes result in high tensile residual stresses in the weld and in the base metal adjacent to it. These stresses are caused by the shrinking of the weld as it cools. They may approach the yield point. Any discontinuity in the weld is also located in this region of high residual stress. The stress rise caused by discontinuity is added to the residual stress caused by welding.

Most fatigue cracking in welded structures initiates at or in the weld. The cracks start at an initial discontinuity in the weld or at the edge of the weld and grow perpendicular to the cyclic stresses. Attachments to girders such as cover plates, stiffeners or other components welded to a flange or web introduce a transverse weld perpendicular to the live load stress thus causing a local stress concentration. This can result in fatigue cracking which starts at the small but sharp discontinuities at the weld toe.

Out-of-Plane Deformations

The type of fatigue described to this point is caused by primary bending stresses. However, the most common type of fatigue cracking in railroad bridges is caused by out-of-plane deformations in girder webs. The case study presented later shows examples of cracking caused by out-of-plane deformations in both thru-plate and deck-plate girder spans and how they were repaired.

In thru-plate girder spans, these deformations result from the rotation of the ends of floor beams. There are many factors that determine the magnitude of this rotational movement, including the stiffness of the floor beam, lateral stiffness of the girder flanges and the details of the floor beam and knee brace connections. At the top of the connection stiffener, the floor beam bracket pulls the web inward and the top flange resists this movement (see Figure 1).

At the bottom of the stiffener near the end supports, the bottom flange is held in place by the bearings. As the girder is rotated by the deflection of the floor beam, the web is deformed between the bottom of the stiffener and the top of the bottom flange (see Figure 2).

In deck-plate girder spans, out-of-plane movements can be introduced at the diaphragms. This can be caused by differential deflection of adjacent beams. The magnitude of the deformation depends on the girder spacing, skew and type of diaphragm. These deformations can also be caused by the strain in the lateral bracing system from the girder deflection or the flange not being fabricated square with the web.
The stress in the web caused by these deformations is an inverse function of the web gap squared. The web gap is the distance from the flange-to-web weld to end of the weld on the stiffener. This is illustrated in Figure 3. These deformations are considered displacement limited. In other words, the characteristics of the span only allow a certain magnitude of deformation. In this case the stress in the web can be reduced significantly by increasing the flexibility of the connection, i.e. increase the web gap by removing part of the stiffener. However, if the deformations are not limited the detail must be made rigid, i.e. connecting the stiffeners to the flange.

Further Discussion

Much of this discussion of fatigue was adapted from information in the National Steel Bridge Alliance document entitled, "A Fatigue Primer for Structural Engineers" authored by John W. Fisher, Geoffrey L. Kulak and Ian F. C. Smith and the paper entitled, "The Evolution of Fatigue Resistant Steel Bridges" authored by John W. Fisher. For a more detailed discussion on fatigue, please reference these documents.

WELDED REPAIRS

General

Welding introduces a more severe initial crack situation than does bolting because of the high tensile residual stresses and inherent discontinuities in the weld. Also, the continuity inherent in welded construction makes it possible for a crack in one element to propagate into an adjoining element unlike a bolted built-up girder. Therefore it is important to be very cautious when using a welded repair. However, I hope to demonstrate that with proper techniques, quality control and design there are cases when welded repairs provide the best solution. What can make a welded repair the best solution? In general, welded repairs are less labor intensive and therefore, more economical. In addition, the structure may not allow the loss of section that is required for a bolted repair. There are many cases where welded repairs are the only practical and economical solution.

To ensure a successful welded repair there are many things that must be considered. Foremost is correcting the condition that led to the cracking. Not only at the cracks but, in all locations in the span. This is typically done by altering the flexibility/rigidity of the detail. However, when changing the local stiffness of the structure, care must be taken to avoid increasing the likelihood of fatigue cracking developing in other locations.

Quality Welding

Several steps are required to ensure a quality weld. The most important step is the development of suitable welding procedures. The welding procedure must include the welding process, the range of base materials to be welded, the filler materials to be used, and the joint designs. Preheat and postheat treatment must be specified including interpass temperatures. For railroad bridges the
American Welding Society Bridge Code D1.5 is used as the guide for development of the welding procedure specifications. Once the welding procedure is approved, the welders must demonstrate that they are capable of performing the required welds. They must either have a current certification or be tested prior to performing the weld.

Finally, inspection is required before, during and after welding. Inspection before welding includes edge preparation, dimensions, cleanliness, root opening, etc. During welding it is important to monitor preheat and interpass temperatures, to observe cleaning between the passes and to watch workmanship of the welded joint. After the weld is completed, an acceptance inspection must be made. This could be done visually or with tests such as magnetic particle, liquid penetrant, radiography or ultrasonic depending on the location and type of weld.

Crack Repair

A typical procedure for a welded crack repair includes crack removal, edge preparation, placement of root, intermediate and final weld passes, grinding of the weld reinforcement flush with the base metal and inspection of the completed weld using ultrasonic inspection technology.

Small cracks are most conveniently removed by mechanical means, such as a grinding wheel or a high-speed rotary tungsten carbide burr, which also prepares the edge for welding. This process is too time consuming for larger cracks where air carbon arc gouging should be used. The base metal should be preheated prior to air carbon arc gouging. After the crack has been gouged, the groove should be cleaned by rotary disc grinding providing a groove with the required radius and angle. Finally, after cooling the groove and groove edges should be visually inspected and a magnetic particle test performed.

Prior to welding the base metal must be preheated. Visual inspection of each weld pass should be made, removing any slag inclusions and repairing any underfill. After the weld is completed, the weld should be ground flush with the base metal. This process is then repeated for the other side of the plate.

After the entire crack is repaired, it should be inspected with the use of ultrasonic equipment. Repairs should be gouged out and rewelded if they don't satisfy AWS D1.5.

Other Considerations

Other things to be considered while evaluating the applicability of a welded repair include the weldability of the material in question and the condition of the surface to be welded. Most low carbon steels are weldable, as are some forms of gray cast-iron. White cast-iron and wrought iron are unweldable. Also, welding to a corroded surface typically leads to a poor weld.

Typically, welding in the compression zone is not a problem unless a condition with out-of-plane deformations exists. Common repairs in the compression zone include welding on bearing stiffeners, welding the tops of stiffeners used as connection plates for diaphragms or floor beam brackets to compression flanges and welding shelf angles to the web near the compression flange. Of course all of these repairs can also be done by bolting, but you must check the impact on your net section.
ALTERNATE CRACK REPAIR METHODS

Although welding can be used to repair cracks effectively, there are other methods available. The following is a list of techniques described in "A Fatigue Primer for Structural Engineers" that have been used to successfully repair steel structures containing fatigue cracks.

- Place cover plates on both sides of the cracked plate, bolting them to the plate with high-strength bolts. The plates will provide a load path around the crack. The cause of the crack must be eliminated or mitigated for this to be successful.

- Drill a hole at the end of the crack and fill with high strength bolt. By eliminating the crack tip you greatly increase the force required to propagate the crack.

- Peening the toe of a weld termination that is perpendicular to the stress range is an effective way to prevent small cracks from propagating. Peening introduces compressive residual stresses and also decreases the size of the crack. It effectively increases the fatigue resistance by a category. It has been found to be most effective when performed under dead load, minimizing the possibility of the live load exceeding the compression residual stress from peening.

CONCLUSIONS

Welded repairs to railroad bridges will be successful if:

- Cause of distress is determined and eliminated or mitigated at all similar details.
- A suitable welding procedure is prepared and followed.
- Certified welders trained in the proposed repair technique are used.
- Proper welding inspection is performed.

Before welded repairs should be considered in a tension zone, a fatigue analysis should be performed. A fatigue analysis should also be performed on the as-repaired state to ensure that repairs haven't caused a fatigue problem in another part of the structure.
CASE STUDY

The following case study demonstrates that in the right circumstances welded repairs can be successful.

Introduction

In the spring of 1990, fatigue cracks were found in the webs of welded thru-plate girder (TPG) spans and deck-plate girder (DPG) spans at four bridges on a major railroad line. At the time the cracks were noticed during a routine bridge inspection, these bridges had only been in service for 5-1/2 years.

Description of Bridges

Cracks developed in the webs of welded plate girder spans at the following four bridges:

- Bridge No. 42.74 over Interstate Canal
- Bridge No. 48.32 over State Spur 79A
- Bridge No. 50.22 over County Line Road
- Bridge No. 53.14 over Tri-State Canal, U.S. 26 and B.N., Inc.

Bridge No. 42.74 consists of a single TPG span, Bridge Nos. 48.32 and 50.22 consist of four rolled beam approach spans and one TPG span, and Bridge No. 53.14 consists of six DPG spans and one TPG span. Fatigue cracks were not found in the rolled beam spans.

Typical cross-sections of the TPG and DPG spans are shown in Figure 4. Relevant span data are listed in Table 1. The design and details of these spans were consistent with the industry practice at the time.

Detailed Inspection

Immediately after finding the fatigue cracks, railroad forces drilled holes at the end of the cracks to arrest further propagation. This was followed by a detailed inspection of all welded plate girder spans on the line. The inspection revealed that cracking was occurring in the following locations:

- **TPG spans** - Top of stiffeners supporting floor beam brackets and bottom of stiffeners near the ends of the spans.
- **DPG spans** - Bottom of stiffeners at diaphragm locations.

As expected, the cracks at the top of the stiffeners turned down at angles ranging from approximately 30° to 60°, while the cracks at the bottom of stiffeners turned up at similar angles. Typical cracks are shown in Figures 5 and 6. No propagation of cracking was found in cracks that had been drilled. Cracks were found at the top of 10 stiffeners and at the bottom of 12 stiffeners in the four TPG spans and at the bottom of four stiffeners on the six DPG spans (see Table 1 for location).

The inspection also revealed improper fabrication and poor welding practice at the intermediate stiffeners on the TPG and DPG spans. The following is a partial list of the problems found during the inspection:
The design drawings called for an "each side" fillet weld. However, the stiffeners were welded completely around the bottom and up into the clip of the stiffener at the top.

2. The gap between the top of the bottom flange and the toe of the stiffener called for on the design drawings was violated at most stiffeners. Typically, where a 3" gap was specified, an actual gap of 2 ½" was provided.

3. The welds were oversized at the bottom of the stiffeners. It appeared that each side was welded and then the weld was placed around the toe.

4. At several locations, the web and stiffeners were undercut at both the top and bottom of the stiffener.

5. There were bulbs of weld at the top of many of the stiffeners.

Analysis of Cracking

The traffic on this line consists primarily of 263,000 lb. to 286,000 lb. cars in unit trains. The typical train has approximately 110 cars. A review of the gross tonnage hauled on this line reveals that these structures were subjected to over 5,500,000 axle loads at the time the cracks were noticed, and have been subjected to well over 20,000,000 axle loads since the repairs were made.

Out of Plane Deformations

The fatigue cracks that developed in the webs were caused by out-of-plane deformations. The stress in the web caused by these deformations is an inverse function of the web gap squared. Using the equations in Figure 1 with a 2 ½" web gap, the web stress at the bottom of the stiffener would be approximately 14 ksi. Based on laboratory tests, it is estimated that this stress would produce failure at the stiffener at approximately 4,000,000 cycles, a value not greatly different from the life of the fatigue cracks in the bridges. Increasing the web gap to 4" would decrease the stress to about 5.4 ksi (a value 60% smaller) and with a corresponding life of over 80,000,000 cycles. Thus, flexibility in the web gap is an extremely important factor. The web gap called for on the plans for these bridges was four to six times the thickness of the web, per the AREMA Manual. However, using the minimum web gap specified by the AREMA Manual may not be sufficient for every welded through-plate girder. In this case, the gap was reduced because the fabricator used longer than specified stiffener plates and welded around the bottom of the stiffener and into the clip at the top of the stiffener.

Solutions

As discussed earlier, to be successful, a repair scheme must eliminate or at least mitigate the cause of the distress while, at the same time, restore the structural integrity of the damaged member. Just as important is the need for repairs that can be accomplished within the tight work windows available on high-density railroad lines. The following sections describe the techniques that were employed to successfully repair these bridges.
Reduction of Web Deformation

To minimize deformation of the web at the top of the stiffeners on the TPG spans, the interior stiffeners were welded to the top flange. When the floor beam rotates, the floor beam bracket will be pulling on the flange, rather than the web, thereby virtually eliminating the deformation of the web. All interior stiffeners were welded to the compression flange, even though interior stiffeners that do not support floor beam brackets have much smaller web deformations. It is now common practice for many railroads to weld stiffeners to the compression flange. New provisions in AREMA Chapter 15 now require that interior stiffeners be attached to the top flange.

To minimize deformation of the web at the bottom of the stiffeners on the TPG spans, the weld was ground away from the bottom of the exterior stiffener and enough of the stiffener was removed to allow a 4" web gap (see Figure 7). In addition, extension plates were bolted to the bottom of the exterior stiffeners near the ends of the spans. These plates were then welded to the bottom flange (see Figure 8). Another method could have been to bolt a tee shape to the stiffener and the bottom flange. Our fatigue analysis showed that welding was appropriate in this case.

The American Institute of Steel Construction (AISC) Bridge Fatigue Guide recommends that near the supports, for a distance at least equal to the girder depth, the outside stiffeners should be connected to both top and bottom flanges. This is now an AREMA requirement. One crack was found outside this limit. This suggests that the web gap at the bottom of all outside stiffeners must be large enough to ensure that stresses caused by out-of-plane deformations are within an acceptable level. The lateral stiffness of the girder flange has a great effect on the required gap. The distance from the support that stiffeners should be welded to the top and bottom flanges should be a function of the flange width as well as girder depth.

To minimize deformation of the web at the bottom of the stiffeners on the DPG spans, the weld was ground away from the bottom of the stiffener and enough of the stiffener was removed to create a 4" web gap. Because deformations of the web can only occur at stiffeners that support diaphragms, the repairs were limited to these stiffeners. Although no cracks were found at the top of stiffeners on the DPG spans, all stiffeners supporting diaphragms were welded to the top flange. AREMA now calls for stiffeners supporting diaphragms to be connected to both flanges.

Crack Repairs

To restore the structural integrity of the webs, all cracks were repaired. It was determined that the most cost-effective way to repair the cracks was by welding. To ensure the quality of the welded repairs, the guidelines outlined in the National Cooperative Highway Research Program (NCHRP) Report 321, entitled "Welded Repair of Cracks in Steel Bridge Members," dated October 1989, were followed. The repair procedure for full-depth cracks was as follows:

1. The base metal was preheated to 150° F.
2. The crack was cut out from one side by air carbon arc gouging to approximately half-plate thickness.
3. The groove was cleaned by rotary disc grinding, completing the required groove radius of %" and angle of 20°.
4. After cooling, visual and magnetic particle testing were performed on the grove and groove edges.
5. The base metal in the crack area was preheated to 250°F for welding.
6. The root, intermediate and final weld passes were completed, with visual inspection performed upon completion of each pass.
7. Slag inclusions were removed and weld underfill repaired.
8. Weld reinforcement was ground flush with the base metal.
9. The area was post-heated to 400°F for one hour and covered by 6 ¼" thick Owens Corning Fiberglass R-19 insulation for slow cooling.
10. This process was then repeated for the other side of the web.
11. Ultrasonic testing was performed in compliance with American Welding Society (AWS) Bridge Welding Code AWS D1.5-88. The tension member requirements were used.
12. Unsatisfactory repairs were gouged out and rewelded.

Requirements imposed on the welders included:
   a.) The low-hydrogen electrodes had to be dried in an oven prior to use.
   b.) Partially completed welds were kept heated when the web repairs were suspended to allow trains to pass.
   c.) Welds that were started had to be completed the same day.

Partial depth cracks were evaluated individually. The minimum web thickness required to resist shear was determined for each girder. Cracks that did not extend into this minimum thickness were removed by grinding. Cracks that extended into this minimum thickness were repaired with the same procedure used for full-depth cracks.

All repairs were made by railroad welders. Because of the high degree of difficulty associated with making these repairs, the welders were assembled at a railroad yard to practice the procedures prior to travelling to the bridge sites. The practice session was held under the tutelage of Robert W. Hunt Company (Hunt). Hunt also performed all the tests on the welds during the repair. The welders performed beyond expectations. Except for a couple of repairs made on the first day that had to be rewelded because of excessive inclusions, no additional rewelding was required. This not only speaks well for the ability of the welders, but also shows that repairing cracks by welding can be an excellent option.

Conclusions

The repairs made in the summer of 1990 have been subjected to well over 20,000,000 axle loads. No cracking has occurred. The repair methods described in this case study are practical solutions that are cost effective. These bridges experienced heavier loads and more cycles in a much shorter period of time than most welded railroad bridges. Therefore, fatigue cracking was accelerated. There may be many similar bridges in service which will, in the near future, reach the threshold of cycles required to cause fatigue cracking. This case study shows that repairing cracks by welding can be an excellent option.
BIBLIOGRAPHY


## Span Data

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Table 1