# Rehabilitation of Steel Deck Girder Bridges

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ABSTRACT

Two 12-span bridges composed of two steel plate girders with a floor system were constructed in 1960. These were the first all-welded girder bridges built in Wisconsin. Plans were prepared to widen these bridges to carry a third lane and replace the existing deck. Several flaws that required repair were discovered in the welded girders. Before construction a transverse crack in the flange was found under the gusset plate used to attach the lower lateral bracing. The gusset plate was welded longitudinally and transversely to the bottom flange. During construction several other flaws were found: cracks under tack welds on the flanges, fatigue cracks in floor-beam webs, web butt welds with incomplete fusion, poor flange-to-web fillet welds, and improper termination of welded covered plates. These flaws were repaired to extend the life expectancy of the bridge by (a) removing the welded gusset plates, grinding the flange smooth, and bolting on new gusset plates; (b) removing the tack-welded piece and grinding out the cracks in the flanges; (c) removing the stringer-to-floor beam welded web connections, reattaching with bolted angles, and drilling stop holes; (d) drilling stop holes at the flange-web intersections in the tension area and providing bolted splice plates as required; (e) grinding the poor web-to-flange fillet welds into a smooth transition; and (f) placing bolted splice plates across the ends of the welded cover plates. The total rehabilitation was analyzed and found to be cost-effective.


Two bridges carry Interstate 94 across the Wisconsin River. These 12-span bridges are composed of two steel girders with a floor system and were designed in four segments of 3 continuous span units (Figure 1). The bridges were constructed in 1960 and were the first all-welded girder bridges built in Wisconsin. Rehabilitation plans were prepared in 1977 to replace the deteriorated concrete decks and to add four new girders to widen the bridges to three lanes. Lack of funding delayed bidding until September 1981. The contractor, working on an accelerated schedule, completed the contract in December 1983.

In March 1981 bridge maintenance personnel reported that the bottom flange of one of the main girders was cracked, resulting in a 75 percent loss of section. Traffic was rerouted until a bolted splice was placed on the flange. Further investigation revealed that the crack initiated where the bottom lower lateral system was attached to the gusset plate, which was welded to the bottom flange. Although the shop plans showed the gusset plate attached with longitudinal fillet welds, the plate was welded both longitudinally and transversely. The transverse fillet welds were believed to be the initiators of fatigue cracking. Because this detail was repeated at every lower lateral connection, the plans were modified to remove all these welded gusset plates and replace them with bolted plates in the positive moment areas. After the contract had been awarded and rehabilitation work had begun, several more problems with the steel girders were discovered. The repair of the gusset plates and the newly discovered problems are discussed in this paper.

## REHABILITATION ANALYSIS

The original bridges were designed as two steel girders with a floor system because this structural system was thought to be more economical than a multiple-girder system. The bridges were supported on separate piers with a median width of 46 ft. To add an extra lane in each direction, it was decided to build a pier between the existing piers and add four steel girders (Figure 2). The new girders were designed to be the same depth as the existing girders, so that girder deflections would be comparable, although these girders were deeper than required for a multiple-girder system. Since the existing girders were assumed to be in good condition, with only a new deck required, it was determined that the extra cost of building the new girders deeper than required would be muoh less than

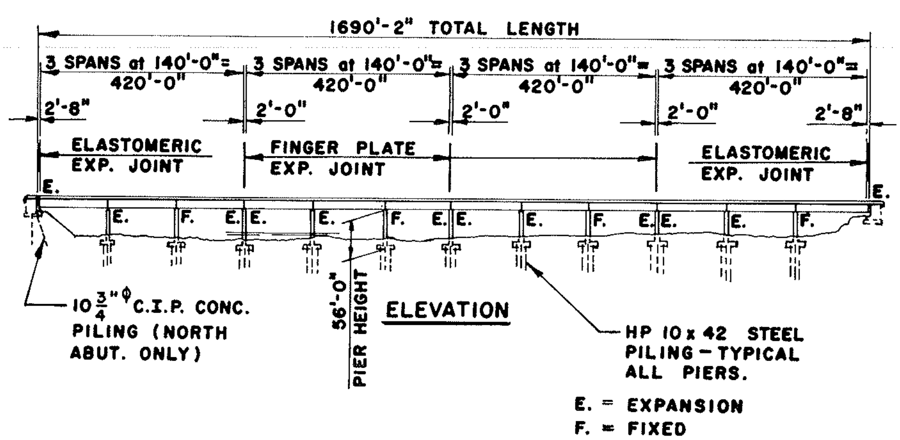


FIGURE 1 Design data.

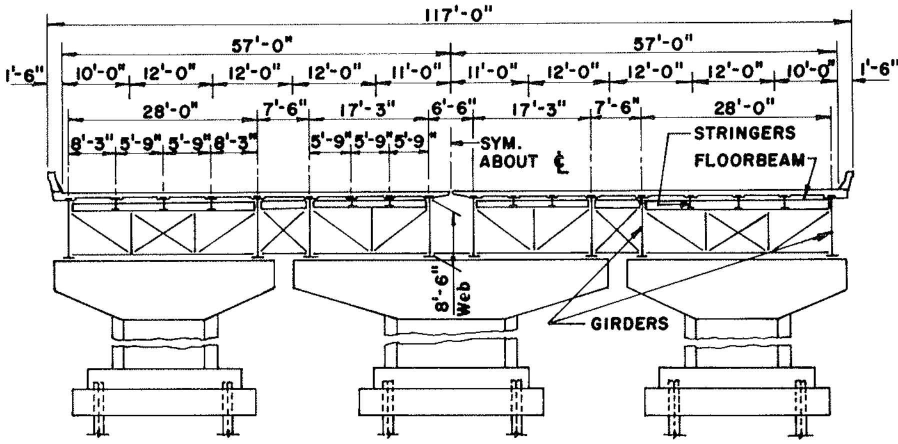


FIGURE 2 Cross section through roadway.

the cost of replacing the existing girders to create a multiple-girder system. The estimated cost to replace the welded gusset plates was considered reasonable and provided effective repair so it was included in the rehabilitation contract.

The properties of the existing bridge steel were evaluated before the contract was let. Charpy energies at 40° F were 80 ft lb for the flange and 56 ft lb for the cover plates. Nil-ductility temperatures were approximately -30° F for the cover plates and -100° F for the flanges. The steel was therefore considered adequate.

The contractor constructed the median piers, new superstructrue steel, and deck before starting deck removal on the existing bridges. As deck removal began on the first bridge, the need for more repair work was discovered. An estimate was made of these additional repairs and contract change orders were placed with the contractor. The following tables show the cost breakdown for the project, including the additional repairs that were required after the original contract was let.

| Item | Cost ($) |
|---|---|
| Contract bid price | 6,057,000 |
| Gusset plates for lower lateral connection | 257,000 |
| Termination of cover plates | 125,000 |
| Top flange connections, flange-to-web welds, weld flaws at web butt weld near flange | 60,000 |
| Floor beam-stringer connections | 450,000 |
| Web cracking at butt welds | 72,000 |
| Ultrasonic testing | 7,000 |
| Substructure repair--pier encasement | 166,000 |
| Bolting cross bracing to existing bridges | 37,000 |
| Total project cost | 7,231,000 |

Superstructure costs (in dollars) were

| | |
|---|---|
| Widening only | 2,315,000 or 28.24/ft² |
| New deck on existing bridge | 792,000 |
| Girder repairs on existing bridge | 1,008,000 |
| | 1,800,000 or 15.20/ft² |

Hindsight indicates that a more thorough inspection of the existing bridges should have been made before the rehabilitation plans were prepared. The estimated final costs of the rehabilitation plans could have been compared to the estimated cost of replacing the existing superstructure with a multiple-girder system, which would be a more redundant structure. This decision would have also had to be weighed against the public response to replacing a 20-year-old bridge.

However, at the time the rehabilitation plans were prepared, there was no reason to suspect the existence of faulty welds and poor construction practices. More and more was being learned about fatigue-prone details but thorough checks for them were just being initiated. Considerable cost and effort were required to repair the existing bridges, but it is believed that fatigue-prone areas have been eliminated or isolated and that these bridges will have a normal service life.

## GUSSET PLATES FOR LOWER LATERAL CONNECTION

The bridges were fabricated entirely from A242 steel. The lower lateral system was attached to the main girders by gusset plates welded to the top side and inside of the bottom flange (Figure 3). The shop details showed the gusset plates to be attached with longitudinal welds only, but the fabricator had welded these plates on all sides. Flange butt welds also occurred under some of these gusset plates. Before the bidding on this bridge was opened, maintenance personnel discovered a crack in the bottom flange in the butt weld under a gusset plate. Ultrasonic testing indicated that the crack reduced the net section by 75 percent (Figure 4). A bolted splice detail (Figure 5) was developed using A588 steel to attach the lower lateral system as well as splice the flanges together. In addition, a hole

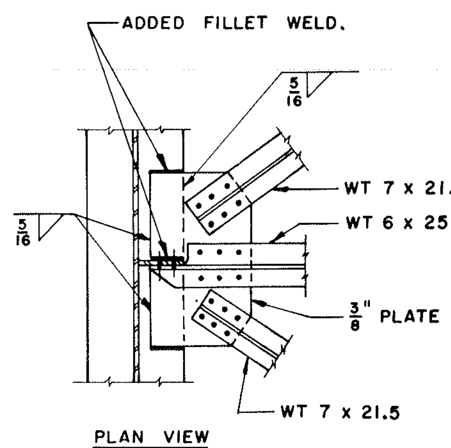


FIGURE 3 Lower lateral connection.

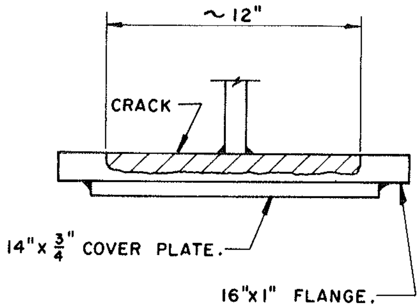


FIGURE 4 Crack location in bottom flange.

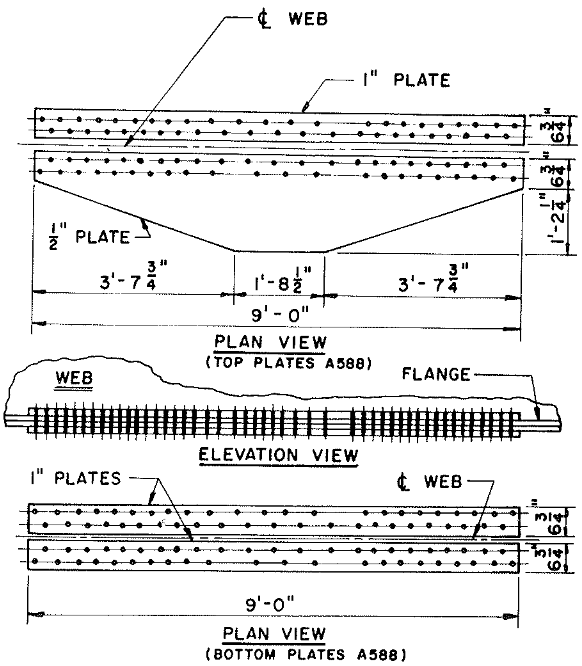


FIGURE 5 Bolted splice detail.

was drilled at the web-flange intersection to prevent crack growth into the web. The fillet welds connecting the cover plate to the flange were removed for 1 in. on both sides of the crack centerline to prevent crack growth into the cover plate.

This repair detail was added to the contract plans. The contractor was required to remove the welded gusset plates, grind the flanges smooth, and ultrasonically test the flange. Where serious flaws were discovered in the flange, full-strength flange splice plates were bolted to the flange. Otherwise only a gusset plate to attach the lateral bracing was bolted to the flange along with a corresponding plate on the bottom of the flange.

In several locations deep gouges (0.5 in. maximum) were discovered in some of the flange plates under the gusset plates. Full-strength flange splice plates were bolted to the flange at these locations. The shallow gouges were ground smooth before the new gusset plates were bolted to the flanges.

Ultrasonic testing was used to determine whether any fatigue-initiating cracks were present after removal of the gusset plates. If no such cracks were found it was assumed that the girder was all right. Where cracks or flaws were detected, the flange plates were isolated from the web by drilling holes, and the splice plates provided the required strength in case the cracks should grow.

Eventually the concrete deck over the first bolted splice repair that had been done 2.5 years earlier was removed. As a follow-up investigation, the splice plates were removed and the flange plates were examined with ultrasonic testing. These tests showed that the initial crack had now grown completely through the flange plate. Because the splice detail was designed for full strength, the plates were replaced and no further action was contemplated because the cracked flange plate now had a full-strength bolted splice.

## TERMINATION OF COVER PLATES

The original plate girders had been fabricated with cover plates welded to the flange plates. This had been done to take advantage of the higher allowable stress for A242 steel with thicknesses of 0.75 in. or less. Because fatigue cracks were not found at the termination of these cover plates but stress levels were high enough to cause them, retrofit plans were prepared to reduce the chance of fatigue cracks starting.

Bolted splice plates were designed to connect both sides of the flange plate at the cover plate termination (Figures 6 and 7). It was assumed that the plane of failure for the flange would occur in this area. These full-strength splices were provided in the tension areas on both top and bottom flange plates to reduce the stresses at these locations and thus provide a longer fatigue life. If fatigue cracks develop in the flange the bolted splice plates will provide the required strength.

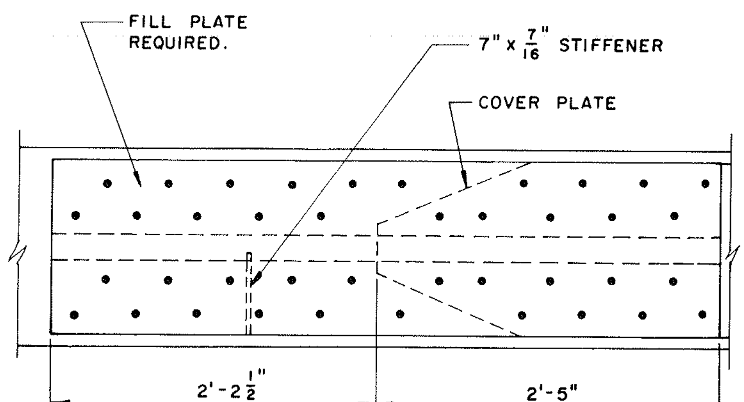


FIGURE 6 Splice detail at end of cover plate.

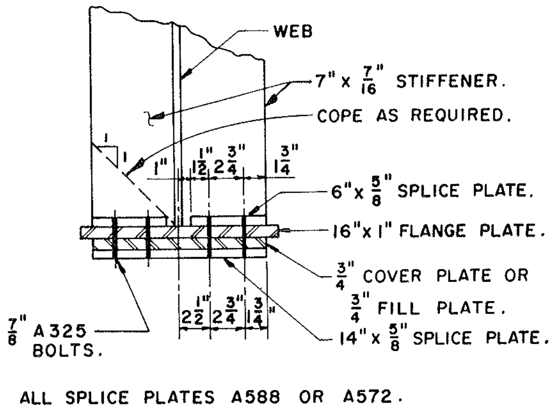


FIGURE 7 Cross section of cover plate.

## TOP FLANGE CONNECTIONS

The bridge was initially built with stay-in-place forms. The contractor used Z-angles at a 4-ft spacing that extended almost all the way across the flange to hold the forms in place. The Z-angles were attached to the flange with four tack welds. Cracks between 1/32 and 1/16 inch deep were discovered under all the welds transverse to the longitudinal axis of the girder flange. These flaws were repaired by removing the angle and grinding the top flange to remove the cracks. Magnetic particle testing was used to check for complete crack removal. These cracks were initially thought to be fatigue cracks but were later assumed to be caused by the tack weld replacement because they occurred under all the tack welds. The cracks also may have been a reaction caused by the galvanizing on the angle or the use of a non-low-hydrogen welding consumable. The original contractor also welded round pipe flanges to the top flange to support the rail for the paving machine. These pipe flanges were removed in the tension areas and the flange was ground smooth. No cracks were found in these areas, probably because more heat was applied to these fillet welds than was used for the tack welds.

## FLOOR BEAM-TO-STRINGER CONNECTIONS

These two-girder bridges have a stringer-to-floor-beam system to support the slab. The 12-span bridge also has 4 units of three continuous spans with expansion joints at the ends of each unit. The stringers were designed as continuous members to cross over the floor beams (Figure 8) except at the ends where they were terminated (Figure 9).

Inspection of these connections revealed fatigue cracks beginning to form in the floor-beam webs where the stringers were continuous over them. These cracks were apparently caused by out-of-plane bending in the web caused by live load stresses. At the end floor beams where the stringers terminated, several fatigue cracks in the web had already propagated to the bottom flange (Figure 10) because there was more deflection occurring here due to lack of restraint because the stringer was not continuous.

To correct these flaws the stringers were removed from the floor beams by flame cutting the stringer flange and web adjacent to the fillet weld connections. The remaining material on the floor-beam web was ground smooth. The stringers were re-attached to the floor beams with clip angles (Figure 11), and studs were placed on the flanges to make the stringers composite with the slab. The composite action reduced the simple span stresses enough to

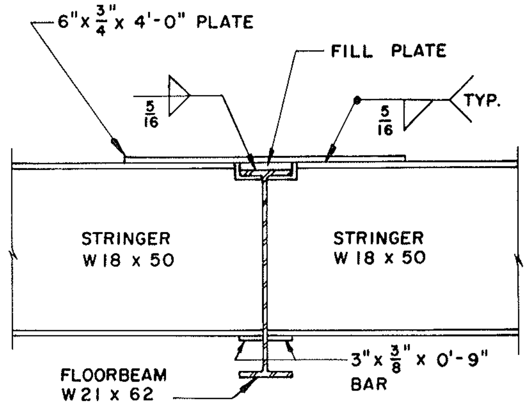


FIGURE 8 Stringer-to-floor beam connection.

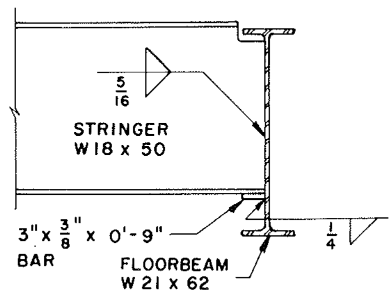


FIGURE 9 Stringer end-to-floor beam connection.

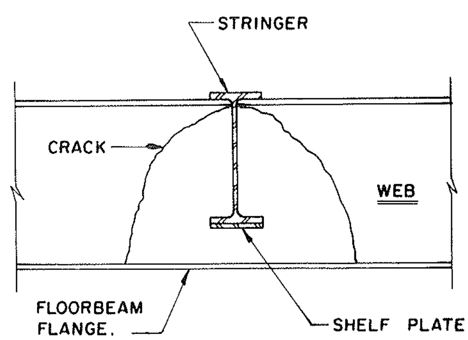


FIGURE 10 Floor-beam web fatigue cracks.

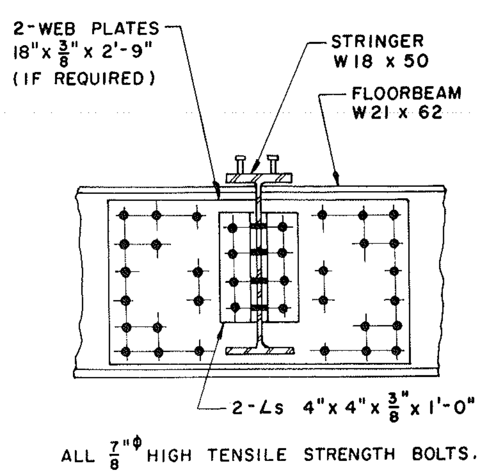


FIGURE 11 Stringer-to-floor beam repair attachment.

compensate for the loss of continuity. Stop holes were drilled at the crack tips on the floor-beam webs.

Even though fatigue cracking had grown more than a few inches into the web, full-bolted web splices were also made. These areas will have to be checked closely in future bridge inspections, but there is redundancy in the number of floor beams to provide a margin of safety.

WEB CRACKING AT BUTT WELDS

Weld flaws were discovered at the web-to-flange intersection where the webs were butt welded together (Figure 12). Apparently the flanges were attached to the web before the web butt welds were made. As a result weld flaws were made during fabrication where the web butt weld terminated near the flange.

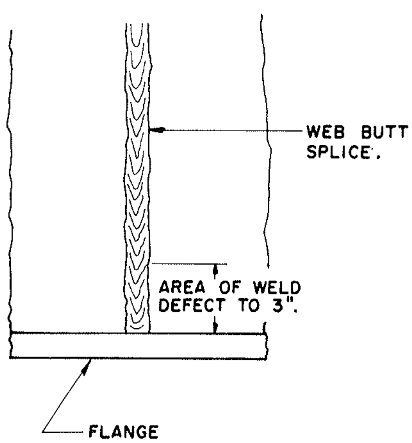


FIGURE 12 Web butt weld flaw.

Two 1-in. round holes were drilled through the web on each side of the weld at the base of the flange (Figure 13). The area between the holes was removed by sawing and the cuts were ground smooth. The portion of the web above the saw cut was checked with dye penetrant to be sure the flaw was completely removed. Further inspection of these web butt welds indicated that some of them had areas with cracks due to lack of fusion at the time of fabrication. A bolted web splice was provided at 33 out of 80 potential locations where the lack of fusion was more than 1 foot long. There was no indication that these flaws were growing but, where no splices were made, they will be checked during future inspections.

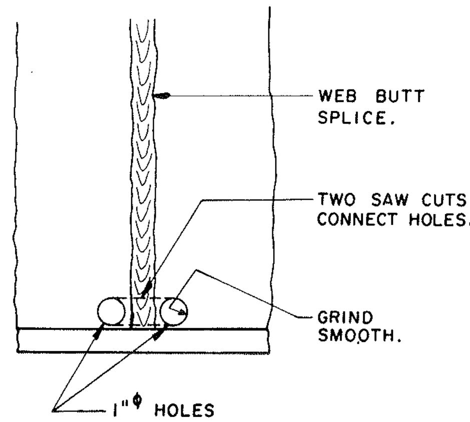


FIGURE 13 Web butt weld repair.

FLANGE-TO-WEB WELDS

Several areas were found where there was undercutting in the flange-to-web fillet welds. These areas were repaired in the tension zones only (top flange over piers and bottom flange in midspan) by grinding the areas with sharp contours. No welding was added because it was believed that there was sufficient weld present and grinding smooth would eliminate any crack initiators.

SUMMARY

After the field flaws were discovered a meeting was held to decide what retrofit action to take. Because the needed repairs were substantial, an estimate was made of the cost of completely removing the floor system and replacing it with two lines of full-depth girders. This estimate was compared with the estimated costs of repairing the existing girders. The cost of replacing the floor system was estimated to be more than the cost of repairing the existing girders ($2,000,000 versus $600,000) so this scheme was not used.

Fatigue cracking was found at the lower lateral gusset connections and at the stringer-to-floor-beam connections. All other cracks and flaws that were found were believed to be caused during initial girder fabrication. Removal or isolation of the fabrication flaws was considered adequate for bridge safety. The locations of the fatigue cracks were also isolated or changed to reduce or eliminate the chances of future crack growth. It is expected that these rehabilitated structures will perform satisfactorily throughout their life.

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