TECHBRIEF

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Fatigue Resistant Rib-to-Floor Beam Connections for Orthotropic Steel Decks with Potential for Automated Fabrication

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Introduction

Orthotropic steel decks (OSDs) have been utilized in many modern bridges. An OSD consists of a continuous steel deck plate on top of longitudinal open or closed ribs that pass through transverse floor beams. All of these components are joined using welded connections. The OSD allows the bridge deck to be integral with the supporting bridge superstructure, resulting in increased rigidity and decreased material use, and it provides a highly redundant structure. The OSD is lighter in weight, is easier to assemble due to its modular nature, and offers a longer service life than other traditional bridge deck systems. These features make it a good choice for long-span, movable, temporary, cable-stayed, and suspension bridges.

Despite the potential advantages of OSDs, one of the barriers to increased use of OSDs in the United States has been the relatively high initial cost of fabrication, resulting from the details specified to achieve the desired fatigue performance of the various welded connections in the deck. Modern OSDs are usually designed with thin-walled closed (U or trapezoidal shaped) ribs and relatively cut-out in the floor beam web. In many cases, an additional, extended cut-out in the floor beam web is located under the rib bottom. In some cases, an internal bulkhead or stiffeners are used within the rib where it passes through the floor beam web.

The rib-to-floor beam (RFB) welded connection is often labor intensive, which adversely impacts the economical fabrication of OSDs. The RFB connection is also fatigue sensitive because it is subjected to complex combinations of stresses from wheel loads on the deck.

The objectives of this research were:

1. to investigate manufacturability of RFB connections, with or without an additional cut-out in the floor beam web under the rib bottom;
2. to investigate available automated (robotic) fabrication processes for these connections;
3. to assess the stress response and the potential for good fatigue performance of identified candidate RFB connections using finite element analysis;
4. to assess the stress response and the potential for good fatigue performance of these connections by full-scale laboratory tests; and
5. to develop recommendations for RFB connection details that have potential for good fatigue performance and are amenable to automated fabrication.

Approach

A review of literature on RFB connections in OSDs was carried out to identify the different types of RFB connections (such as, with or without an additional cut-out and with various cut-out geometries), and the issues related to automated fabrication and the manufacturability of these connections. In addition, information regarding the fatigue performance of these RFB connections, and the factors that affect this fatigue performance were reviewed. This review included published, unpublished, and anecdotal information related to manufacturability and fatigue performance of RFB connections for OSDs.

Factors affecting the manufacturability of RFB connections, including the potential for automated fabrication, were identified. The influences of fit-up gap and fit-up gap tolerance, measurement techniques, weld configurations, cut-out geometries, and the use of internal stiffening on automated fabrication were considered.

In addition, the factors affecting fatigue performance of RFB connections were identified; many of these factors are related to the manufacturability of RFB connections. Several factors affect the stresses from fatigue loading that develop at RFB connections, such as the support/restraint condition of the floor beam, rib depth, floor beam depth, and spacing of floor beams, floor beam web thickness, rib wall thickness, and rib shape. In addition, the welded joint configuration (such as, fillet welded versus partial or full penetration groove welded), as well as fit-up gap and tolerance can affect the fatigue resistance and manufacturability of the connection.

The configuration of the bridge superstructure system can significantly affect the live load stresses that develop at OSD RFB connections. The use of an OSD for a deck replacement application may create OSD support conditions that differ from the OSD support conditions for a new bridge application

The geometry of OSDs used in deck replacement applications is constrained significantly by the existing superstructure components. Frequently, relative to a new bridge application, the rib spans are longer (dictated by the existing floor beam (or floor truss) spacing), the depth of the OSD floor beam is limited by the distance from the top of the existing floor beam (or floor truss) to the roadway, and some of OSD floor beams are located on top of existing floor beams (or floor trusses) of the superstructure. In these applications, the OSD floor beam is supported by the existing floor beam (or floor truss), which tends to decrease stresses in the OSD floor beam from in-plane actions, such as in-plane shear. At the same time, however, the bottom flange of the OSD floor beam is also restrained transversely by the existing floor beam (or floor truss), which tends to increase stresses at the RFB connection from out-of-plane (i.e., out of the plane of the floor beam web) rotation demands on the RFB connection imposed by the ribs from primary bending of the ribs as they deflect under wheel loads within the rib span between adjacent floor beams.

For OSDs in new bridge applications, the bridge and OSD can be designed simultaneously to balance the important OSD geometric parameters, such as rib span and floor beam depth, with the remaining superstructure geometric parameters. New bridge applications permit the use of independent floor beams that are not restrained by other transverse members in the bridge. An independent OSD floor beam is subject to larger in-plane shear forces, however, rib rotations produce less out-of-plane bending stress in the floor beam web because the unrestrained floor beam bottom flange allows the RFB connection to rotate out-of-plane more freely.

Candidate RFB connections were identified that have potential for good fatigue performance and are amenable to automated fabrication. Parametric finite element analyses (FEA), which varied selected geometric factors, were conducted on a simple steel bridge superstructure sub-assembly containing an OSD with the candidate RFB connections. The three-dimensional (3D) FEA sub-assembly models of the connections were subjected to the fatigue loading for OSDs specified in the AASHTO LRFD Bridge Design Specifications (BDS) (AASHTO 2016), to evaluate stresses and assess the potential for good fatigue performance of the candidate RFB connections.

The candidate RFB connections that were selected for laboratory tests are the fitted RFB connection and the slit RFB connection (see Figure 1). Four full-scale test specimens were designed, two with fitted RFB connections and two with slit RFB connections (i.e., RFB connections with a slit cut-out). Each specimen includes a deck plate, one floor beam, four ribs, and an edge plate girder. These test specimens were initially assessed using FEA.

The test specimens were tested under static and

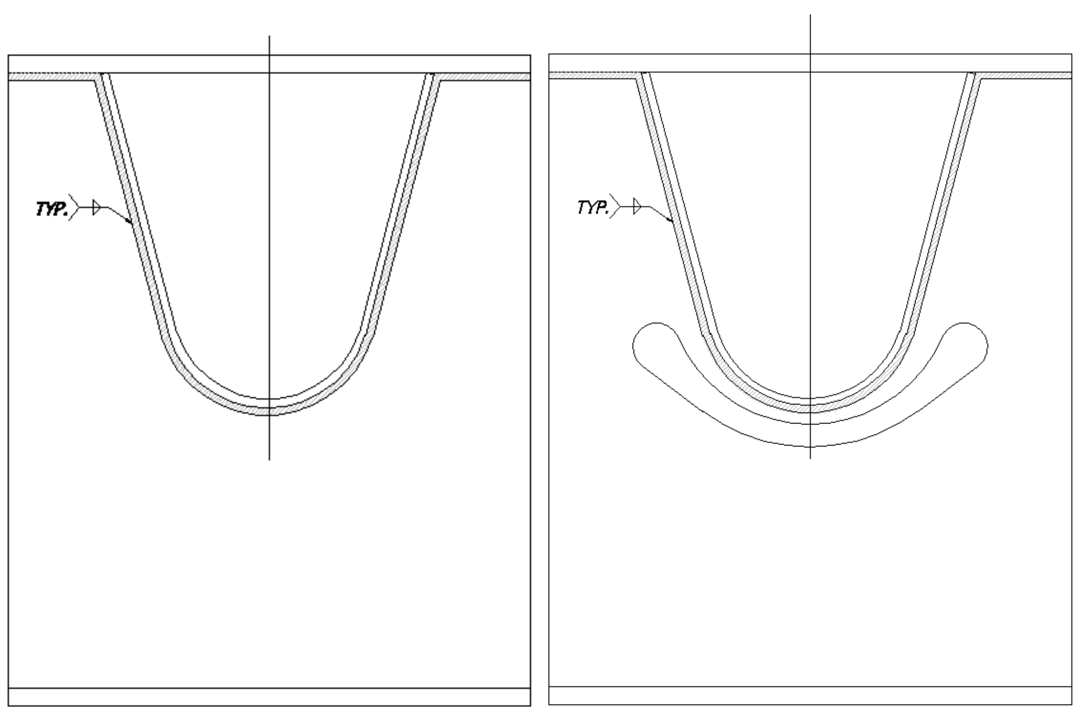

Figure 1. Fitted RFB connection (left) and slit RFB connection (right)

fatigue loading (see Figure 2) conforming to the AASHTO BDS fatigue loading for OSDs (AASHTO 2016). Loading protocols were developed to produce fatigue stresses at RFB connections from floor beam in-plane response, and floor beam out-of-plane response (i.e., from rib rotation) as determined from FEA. The specimens were instrumented to measure strains and displacements at fatigue prone locations to quantitatively assess the fatigue stresses and compare with the FEA results.

Figure 2. Slit RFB Connection test specimen

Based on the research results, recommendations were developed for RFB connection details with potential for good fatigue performance and are amenable to automated fabrication. Suggestions were developed for suitable automated fabrication techniques for RFB connections.

Results

Finite Element Analysis

FEA was performed using a model of a bridge with an OSD termed the "plate girder" (PG) model (Figure 3). The model consists of five floor beams, two I-shaped edge plate girders, and eleven U-shaped ribs. The floor beams span 25 feet-8 inches between the edge plate girders. The center-to-center rib spacing is 28 inches, rib depth is 14 inches, rib thickness is 5/16 inch, and rib bend radius is 4 inches. The floor beam bottom flange thickness is ¾ inch and the deck plate thickness is ¾ inch. Different floor beam web depths, floor beam web thicknesses, floor beam spacing, and floor beam restraint conditions were used to study their effects on RFB connection stresses. Two different center-to-center spacings of the floor beams were used. One spacing was 11 feet-9 inches, which was used to study in-plane loading and response of RFB connections, while the second spacing of 20 feet was used to study out-of-plane loading and response of RFB connections. The 20 feet spacing is the maximum spacing recommended by the FHWA Manual for Design, Construction, and Maintenance of OSD Bridges (Connor et al.,

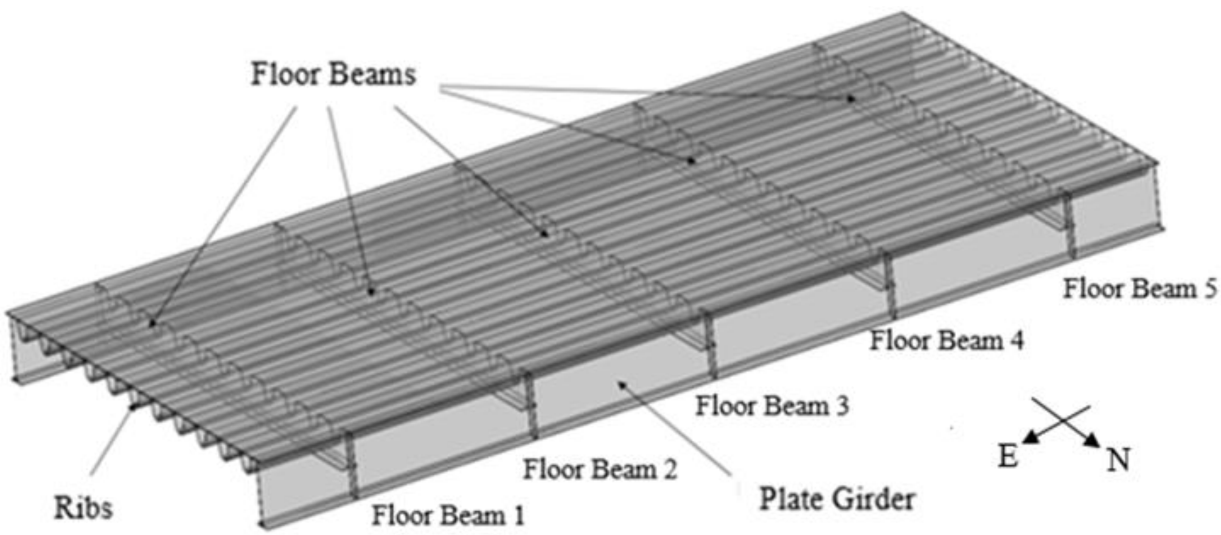

Figure 3. Overview of the plate girder model showing plate girders, ribs, floor beams and OSD

2012). In-plane loading is defined as loading that produces largely in-plane stress response (in the plane of the floor beam web), such as in-plane shear in the floor beam, while out-of-plane loading is defined as loading that produces largely out-of-plane response, such as rib rotation.

The vertical support and transverse restraint of an OSD floor beam affects the behavior of the RFB connection. The condition where the OSD floor beam has no support below the bottom flange and is unrestrained between plate girders (as shown in Figure 3) is referred to as an independent floor beam. The condition where the OSD floor beam is restrained by larger transverse elements in the bridge (trusses or large transverse floor beams) is referred to as a restrained floor beam. To study the effect of the restrained floor beam condition on the stress response of RFB connections, the restraint provided by a large transverse floor truss was implemented into the FEA model.

Fitted RFB Connection FEA

The fitted RFB connection was analyzed under in-plane loading using the independent floor beam condition. The total factored full tandem axle load specified by the AASHTO BDS for the Fatigue I load case, equal to 83 kips, was applied. A loading condition using the full tandem axle load was considered which results in high stresses at the RFB connection from in-plane loading. In this condition, the total factored full tandem axle load is centered on Floor Beam 3 of the PG model (see Figure 3) with one half of the full tandem (two load pads) centered between Rib 1 and Rib 2.

Three stresses were evaluated at Rib 1; (1) the stress normal to the RFB connection weld toe on the rib wall; (2) the stress normal to the RFB connection weld toe on the floor beam web and; (3) the stress tangent to the weld axis at the RFB connection weld root. As shown in Figure 4, the largest stress normal to the weld toe on the floor beam is below the 10 ksi Constant Amplitude Fatigue Limit (CAFL) for AASHTO Fatigue Category C, the AASHTO fatigue category appropriate for this detail (AASHTO 2016), and therefore this weld toe is expected to exhibit good fatigue performance.

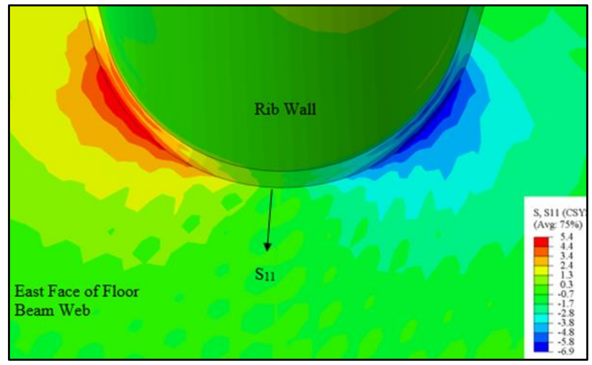

Figure 4. Contour plot of stress normal to weld toe on floor beam web of fitted RFB connection

A contour plot of the stress normal to the RFB connection weld toe on the rib wall for FBS1 is shown in Figure 5.

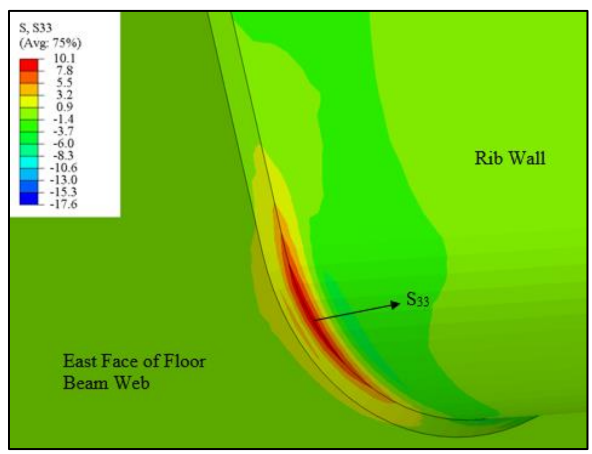

Figure 5. Contour plot of stress normal to weld toe on rib wall of fitted RFB connection

As shown, the largest stress is just above the 10 ksi CAFL, and therefore this weld toe may not exhibit good fatigue performance. The largest stress at this weld toe is driven by shear force in the floor beam web, which is large near Rib 1. The floor beam web response to the shear force distorts the rib opening in the web, resulting in local rib wall bending. Since this stress is above the CAFL, a study on the effect of the floor beam web geometry was performed.

Slit RFB Connection FEA

The slit RFB connection was studied with an independent floor beam under in-plane loading. The floor beam spacing was 11 feet-9 inches. The loading configuration was similar to that used in the fitted RFB connection in-plane loading study. The FEA results for the slit RFB connection with an independent floor beam show large magnitude principal stresses around the slit edges of the Rib 1 RFB connection, as shown in the left side of Figure 6. A large tension principal stress of 27.1 ksi can be seen on the lower south edge of the slit, which is adjacent to the plate girder. This stress is a result of a diagonal tension stress field in the floor beam web, which is driven by shear force in the floor beam. This large principal stress is larger than the 24 ksi CAFL for AASHTO Fatigue Category A, the AASHTO fatigue category appropriate for this detail (AASHTO 2016), and is unlikely to exhibit good fatigue performance. The largest tension stress normal to the weld toe on the rib wall is 3.1 ksi and the largest tension stress normal to the weld toe on the floor beam web is 2.9 ksi, both significantly less than the 10 ksi CAFL for AASHTO Fatigue Category C. The presence of the slit significantly decreases the stresses at the RFB connection welds, but the stress at the slit edge is large.

The slit RFB connection was also investigated with a restrained floor beam under in-plane loading, to study the effect of restraining the floor beam and reducing the shear force in the OSD floor beam. The floor beam spacing was 11 feet-9 inches. By restraining the floor beam and reducing the OSD floor beam shear, the tension principal stress in the floor beam web at the slit edge is significantly reduced. As shown in the right side of Figure 6, the largest tension principal stress is 15.5 ksi at the top of the north side of the slit. At the bottom south side of the slit, the largest tension principal stress is 9.8 ksi, indicating a 64% decrease. These stresses are less than the 24 ksi CAFL for AASHTO Fatigue Category A, therefore, the slit RFB connection with a restrained floor beam is likely to exhibit good fatigue performance under in-plane loading.

Full-Scale Fatigue Testing

Four full-scale RFB connection fatigue test specimens, two with fitted RFB connections and two with slit RFB connections, were designed and fabricated. The fitted RFB connection was investigated as it was considered amenable to automated fabrication, and the FEA results for

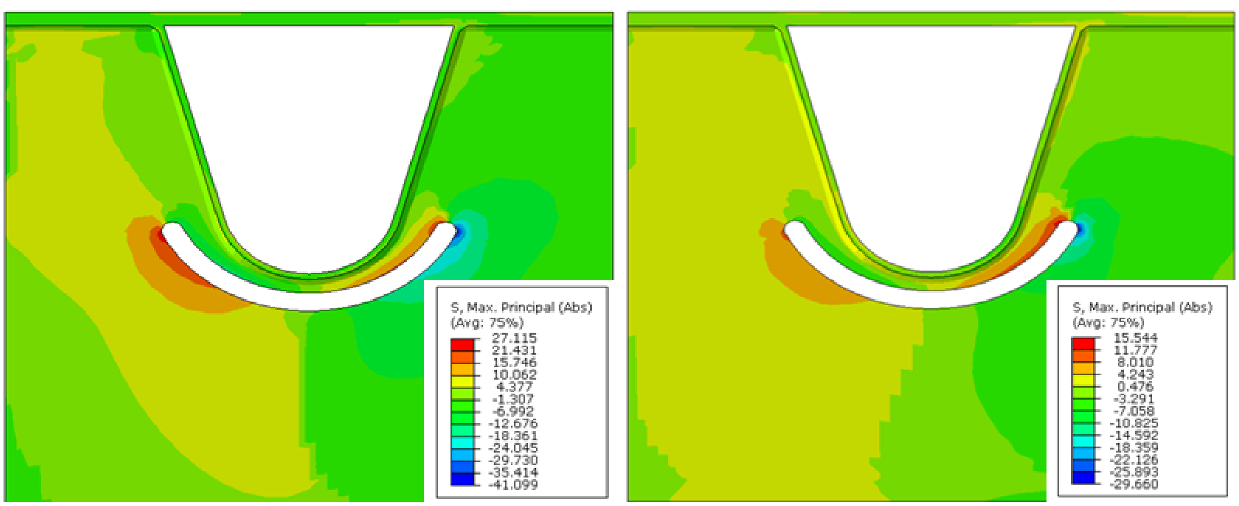

Figure 6. Contour plot of principal stress on floor beam web at slit RFB connection with a restrained (left) and unrestrained (right) floor beams.

an independent floor beam indicated potential for good fatigue performance. The slit RFB connection was investigated since fabrication is similar to fitted RFB connection fabrication, FEA results for a restrained floor beam indicated the potential for good fatigue performance, and this type of RFB connection has not been tested before.

Each of the four test specimens consists of a rib-deck-floor beam panel to which an edge plate girder is attached to one end of the floor beam (Figure 7). Attached to the other end of the floor beam is an extension beam, which enables the transverse load position (along the floor beam) to be varied. The rib-deck-floor beam panel includes: four 6 feet long, 5/16 inch thick cold-bent ribs; a 6 feet by 10 feet-4 inches, ¾ inch thick deck plate; and a built up 9 feet-11 inches long, 2 feet-4 inches deep floor beam. The floor beam web is 26 and ½ inches in depth and ½ inch thick. The floor beam bottom flange is ¾ inch thick.

The test specimens were manufactured using an automated fabrication process. To automate the process of fitting the floor beam web plates to the assembled rib-deck panels, laser tracker measurements of the rib-deck panels were made and the measurements were used to match-cut the web plates. The ribs were positioned on the deck plate according to the design drawings and were welded to the deck plate. Then, 2-dimensional measurements of each rib-deck panel were made using the laser tracker. The centerline location of the floor beam web plate on the assembled rib-deck panel was marked. Laser measurements along the centerline provided the as-built profile of the rib-deck panel, which the floor beam web plate should match to minimize the fit-up gap. The laser measurement data for the assembled rib-deck panels were used as input to a plasma cutting machine to match-cut the floor beam web plates. The match-cut web plates were then fit to the rib-deck panels. Some manual grinding was required to complete the fit-up. After fit-up, the web plate was tack welded to the bottom of the deck plate and the ribs. The fit-up process could be improved to eliminate manual grinding by cutting the floor beam webs with greater precision, for example, using a CNC machine, as opposed to the more common plasma cutting.

Measurements of the fit-up gaps between the floor beam web plate and ribs were made. The maximum fit-up gap among the four ribs for all four test specimens was 0.094 inch. The minimum fit-up gap was 0.020 inch. Two previous trial specimens were fabricated according to the dimensions on the design drawings and the maximum fit-up gap for these trial specimens was ¼ inch. By automated match-cutting the web plates to fit the measured as-fabricated rib-deck panels, significantly smaller fit-up gaps were achieved.

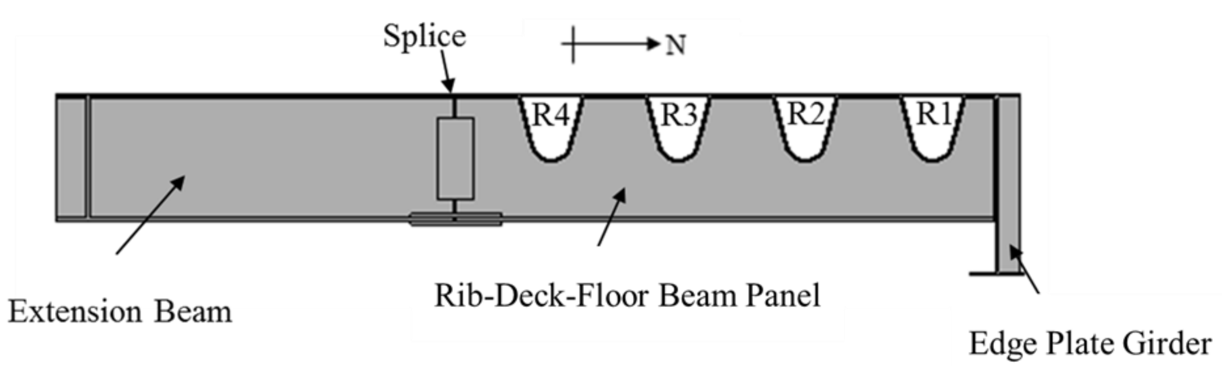

Figure 7. Transverse view of fitted RFB connection test specimen with extension beam (note: slit RFB connection test specimen is similar)

For all four specimens, the RFB connection weld was a doubled sided, 5/16 inch fillet weld. The two slit RFB connection specimens were manually welded using the gas metal arc welding (GMAW) process. These welds were not expected to govern the fatigue response of the slit RFB specimens. For the two fitted RFB connection specimens, the RFB connection welds were made with a robotic welding process. The welding was performed with the deck panel in the inverted position. The RFB connection welds were made as near-vertical welds (upward) using a GMAW welding process with a flux core welding wire.

Slit RFB Connection Testing

As observed from FEA of the slit RFB connection under out-of-plane loading, the stress demands are small at the welds and the slit edge. Since these stresses were not expected to be large in the test specimens, the slit RFB connection specimens with restrained floor beams were tested statically to verify that low stresses develop under large rib rotations from out-of-plane loading. The slit RFB connection specimens were tested using hydraulic actuators to generate rib rotations up to 36% larger than the rib rotations observed from FEA of the PG model under out-of-plane loading. A load case that produced these large rotations was used to verify that the stress demands from out-of-plane loading are not a fatigue concern for the slit RFB connection. A restraint fixture, simulating the longitudinal and transverse stiffness provided by the transverse floor truss in the FEA model, was used to restrain the floor beam of the slit RFB specimens.

Strain gages were installed on the rib walls and floor beam webs of the slit RFB connection specimens. Gages were installed normal to the weld toes of the RFB connection and on both the top and bottom of the slit cut-out edge. The largest stress ranges observed normal to the RFB weld on the floor beam web and on the rib wall were 1.0 ksi and 9.5 ksi, respectively, which are less than the 10 ksi CAFL for AASHTO Fatigue Category C. The largest stress range observed on the slit edge was 12.4 ksi, which is much less than the 24 ksi CAFL for AASHTO Fatigue Category A. As expected, even with rib rotations that are larger than those observed from FEA of the PG model, the corresponding stress ranges are unlikely to cause fatigue cracking.

Two different cyclic in-plane loading configurations were used for fatigue testing of the slit RFB connection test specimens to maximize in-plane response and understand the fatigue performance. One loading configuration, termed Phase A, used the total factored full tandem axle load of 83 kips, and was located transversely to maximize the in-plane response of the slit RFB connections at Rib 1 and Rib 4. The longitudinal center of the tandem axle was centered on the floor beam web. A second load configuration, termed Phase B, was located transversely to maximize the in-plane response of the slit RFB connections at Rib 2 and Rib 3. The load was centered on the floor beam web. The load range for the Phase B configuration was 25% larger than the total factored full tandem axle load, and was 104 kips. Similar to the out-of-plane loading test, a restraint fixture simulating the transverse floor truss in the FEA model restrained the floor beam in the in-plane loading tests.

During Phase A of testing, a total of 2.6 million cycles were applied to the two slit RFB specimens and no fatigue cracks were observed. At the slit RFB connection fillet welds, the stresses at the floor beam weld toe and the rib wall weld toe were relatively small in magnitude. The largest stress range for both specimens was at the rib wall weld toes for Rib 1 and Rib 4, with a stress range of 5.5 ksi, which is much less than the 10 ksi CAFL for AASHTO Fatigue Category C. At the slit edge at Rib 1, the largest stress range was 17.8 ksi in tension, and 31.6 ksi in compression. The compression stress range is larger than the 24 ksi CAFL for AASHTO Fatigue Category A. These stresses develop from a combination of behavior, including the floor beam shear passing around the slit toward the edge girder, the reaction in the floor beam web to torsion in the rib caused by eccentricity of the applied load relative to the rib, and the floor beam web reactions to the rib wall shear forces. These mechanisms combine to cause the slit to locally "close", causing large compression stresses at the slit edge. This stress range was tested beyond the design fatigue life for AASHTO Fatigue Category A. No fatigue cracks were observed and therefore, the slit RFB connection exhibited the potential for good fatigue performance during Phase A testing.

During Phase B of testing, a total of 2.3 million cycles were applied to the two slit RFB specimens and no fatigue cracks were observed. The largest stress range at the rib wall weld toe and floor beam weld toe was 6.6 ksi at Rib 3. At Rib 2, the largest stress ranges at the slit edge were 16.9 ksi in tension, and 34.9 ksi in compression. No fatigue cracks were observed and therefore, the slit RFB connection exhibited the potential for good fatigue performance during Phase B testing.

Fitted RFB Connection Testing

One phase of cyclic in-plane loading fatigue testing of the fitted RFB connection test specimens was performed. Under in-plane loading, the measured responses of the fitted RFB connections were generated by the effects of floor beam web shear and local rib wall bending. The load was located transversely to maximize the in-plane response of the fitted RFB connections at Rib 1 and Rib 2. The load was centered on the floor beam web. The load range was 25% larger than the total factored full tandem axle load, and was 104 kips. This larger load range was used to generate rib wall weld toe stresses significantly greater than the 10 ksi CAFL for AASHTO Fatigue Category C, so fatigue cracking would be likely. A floor beam restraint fixture was not used in order to simulate an independent floor beam condition. Strain gages were installed normal to the rib wall weld toe and floor beam weld toe at the RFB connection of each of the four ribs.

A total of 1.97 million cycles were applied to the two fitted RFB connection test specimens. Fatigue cracks were found in the RFB connection welds of Rib 1 and Rib 2 in both test specimens. The largest stress range normal to the rib wall weld toe in tension was 17.2 ksi at Rib 1. The largest stress range normal to the rib wall weld toe in compression was 22.4 ksi at Rib 2. Although both of these largest stress ranges are much greater than the 10 ksi CAFL for AASHTO Fatigue Category C which is applicable to the weld toe, the fatigue cracks initiated from the weld root (Figure 8). The fatigue cracks initiated from regions of the weld that were subjected to tension stress ranges, as well as regions of the weld that were subjected to compressive stress ranges. Fractographic examination of the fracture surface was performed to confirm that the fatigue cracks initiated from the weld root (Figure 9).

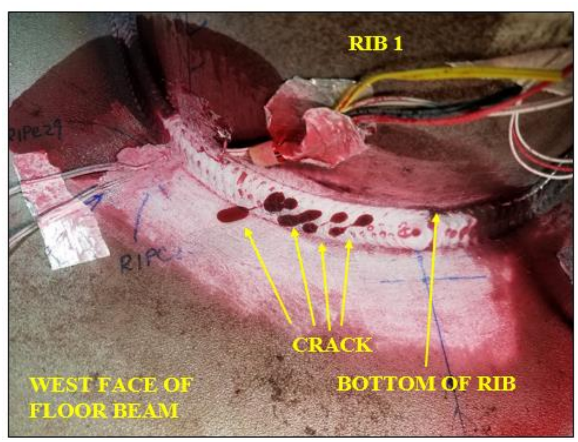

Figure 8. Cracks in west fillet weld at north side of fitted RFB connection Rib 1

The Palmgren-Miner fatigue damage index (Miner 1945) was applied to the fitted RFB connection test specimens to compare the potential for weld toe cracking with the potential for weld root cracking during the in-plane loading fatigue testing. This fatigue damage index is a sum of the number of cycles at a given stress range (n) divided by the fatigue life at that stress range (N) over various stress ranges applied to a fatigue detail. A fatigue damage index of 1.0 (or greater) indicates that the fatigue life is reached (or exceeded), and fatigue cracking is expected.

The Palmgren-Miner fatigue damage index was calculated for rib wall weld toe cracking on the north side of the bottom of Rib 1 and Rib 2 of the RFB test specimens based on stresses from FEA. The mean S-N curve for AASHTO Fatigue Category C was used to determine the fatigue life at the stress range determined from FEA. Fatigue damage indices exceeding 1.0 are calculated for locations with circumferential angles ranging from 35 to 75 degrees from the bottom of the rib, with a maximum fatigue damage index of approximately 2.3. The fatigue damage indices for Rib 2 are smaller, but exceed 1.0 at locations with circumferential angles ranging from 45 to 70 degrees from the bottom of the rib. These results are consistent with the anticipation of rib wall weld toe fatigue cracking during the tests.

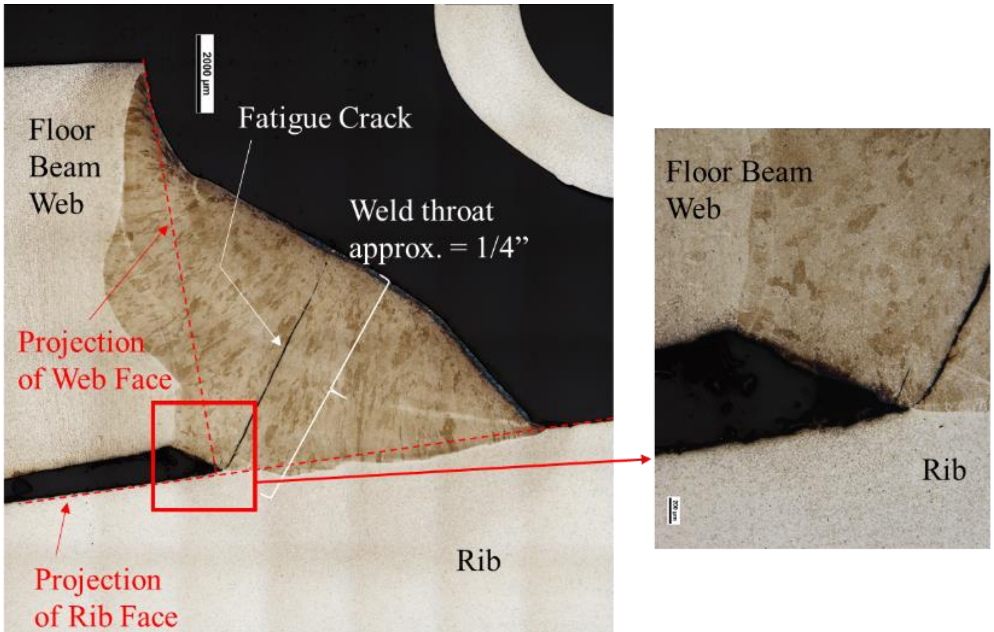

Figure 9. Light optical microscopy images of polished and etched surface perpendicular to weld axis, showing cracks initiated from weld root

Similarly, the fatigue damage index was calculated for weld root cracking of the RFB test specimens based on stresses from FEA. The AASHTO BDS have no specific provisions for fatigue evaluation of the weld root of an RFB connection. However, fatigue cracking under normal stress tangent to the weld root (i.e., the tangential stress, tangent to the weld axis) would develop and propagate in a plane perpendicular to the weld axis. Therefore, to determine the fatigue damage index for weld root cracking, this tangential stress was used along with the mean fatigue resistance for AASHTO Fatigue Category B, which appears to be appropriate for this type of fatigue cracking. The fatigue damage indices for both Rib 1 and Rib 2 are less than 1.0, indicating that root fatigue cracking is not anticipated. However, weld root fatigue cracks were observed in the test specimens, which suggests that predicting weld root cracking using only the tangential normal stress from a simple FEA model and the fatigue resistance for the corresponding AASHTO Fatigue Category B is ineffective. Other contributing factors, such as shear stress within the weld, and the potential for variation of root discontinuities around the rib bottom may be important.

Cyclic out-of-plane loading fatigue testing of the fitted RFB connection test specimens was performed. One specimen was tested by imposing rib rotations similar to those observed from FEA of the PG model under out-of-plane loading, and the second specimen was tested by imposing rib rotations that were 25% larger than observed from the FEA results. A restraint fixture simulating the transverse floor truss in the FEA model restrained the floor beam in the fatigue tests.

A total of 4.77 million cycles were applied to the RFB connection specimen subjected to rib rotations similar to those from the FEA. A total of 1.34 million cycles were applied to the test specimen subjected to the 25% larger rib rotations. The stress ranges at the bottom of the rib normal to the floor beam weld toe were carefully monitored, as this location is where fatigue cracking was expected for out-of-plane loading. The number of fatigue cycles applied to both specimens exceeded the mean and upper bound fatigue life for AASHTO Fatigue Category C at these stress ranges. No fatigue cracks were observed in either specimen. These results indicate that restrained floor beams with fitted RFB connections have the potential for good fatigue performance. In addition, a fillet weld stop at the rib bottom, created during fabrication of the fitted RFB connection specimens, where stresses normal to the floor beam web weld toe are largest during out-of-plane loading, does not appear to have a detrimental effect on observed fatigue performance.

Conclusions

The following conclusions are drawn from the results of this research:

- Automated fabrication techniques including laser measurements, match cutting of floor beam webs, and robotic welding were used successfully to fabricate the test specimens and are promising techniques for improving the manufacturability of OSDs.
- Match cutting the floor beam web plates to fit the as-fabricated profile of the rib-deck panel at the locations of the floor beams, based on measurements from a laser tracker, was shown to be an effective process for creating tight fit between the floor beam web plates and ribs, without significant manual grinding of the floor beam web plates.
- Robotic welding was shown to be an effective process for making fillet welded RFB connections of the floor beam web to the assembled rib-deck panel in the inverted position (i.e., deck surface down, rib bottom up) without rotating the panel, including welding in the upward (opposite the direction of gravity) direction.
- Robotic welding and other automated fabrication techniques to improve OSD manufacturability can be easily applied to slit RFB connections, since the slit cut-out is located entirely within the floor beam web, which enables a continuous RFB connection fillet weld to be made.
- Based on the global FEA results and laboratory testing results under static and cyclic loading, the slit RFB connection with a restrained floor beam demonstrated good fatigue performance. This RFB connection type is recommended for use in deck replacement applications (i.e., restrained floor beam configurations).

- Based on the FEA results, the slit RFB connection does not appear to be appropriate for use in new construction applications (i.e., independent floor beam configurations).

- For the fitted RFB connection test specimens, fatigue assessment of the weld root using only normal stresses from FEA indicated that weld root cracking was not expected at the locations where root cracking was observed in the laboratory tests; however, weld toe cracking was expected, but was not observed. Therefore, predicting weld root cracking using only the normal stresses from FEA appears to be ineffective.

- Based on FEA results, the fitted RFB connection with a restrained floor beam appeared unlikely to have good fatigue performance for out-of-plane loading unless the out-of-plane flexibility of the web was increased, however, based on laboratory testing under cyclic out-of-plane loading, the fitted RFB connection with a restrained floor beam demonstrated good fatigue performance.

The following recommendations are based on the research:

- Laser measurements of the as-fabricated rib-deck panel profile at the locations of the floor beams and automated match cutting of the floor beam web plates to the as-fabricated profile should be implemented in OSD fabrication to improve the fit-up of the floor beam webs to the ribs, and to minimize manual grinding.

- The fitted RFB connection type should be considered as a preferred design option for new construction applications, where the OSD floor beams are unrestrained, due to its potential for automated fabrication and for good fatigue performance.

- For OSDs in deck replacement applications, where the floor beams of the OSD may be restrained by the existing superstructure, the slit RFB connection type should be considered as a design option due to its potential for automated fabrication and for good fatigue performance.

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Fatigue Resistant Rib-to-Floor Beam Connections for Orthotropic Steel Decks with Potential for Automated Fabrication

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