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Design and Fabrication of Orthotropic Deck Details

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16. Abstract

The objectives of the research were to verify the design and fabrication of the orthotropic deck details proposed for the lift bridge, for infinite fatigue life. Multi-level 3D finite element analyses (FEA) of the proposed deck were performed to determine the critical stresses at the connections, the corresponding load position, and the deck specimen. To develop cost-effective connection details, three variations of rib-to-floor beam and rib-to-deck plate connection details, including the influence of different fabrication parameters, were explored in full-scale small size mockups. Subsequently, the infinite life fatigue performance of the connection details were evaluated by laboratory testing of a full-scale prototype. The fatigue testing was conducted under simulated rear tandem axle loading of the AASHTO fatigue truck with adequate boundary condition. The prototype testing was runout after 8 million cycles, verifying the infinite life fatigue performance of the deck design.

17. Key Words Lift Span, Orthotropic Deck, Rounded Bottom Ribs, Infinite Fatigue Life, Full-Scale Prototype, Fatigue Testing, AASHTO, Run-Out, Cost-Effective, Fabrication.	18. Distribution Statement

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10. POST-TEST DESTRUCTIVE EVALUATION

The activities undertaken in Task 10 of the research project, related to the post-test destructive evaluation of the rib-to-floor beam welded connection, are reported in this chapter. Multiple polished and macro-etched cross sections of Rib 1-to-inner floor beam connection were prepared at discreet locations, and detailed measurements of the welds were made at each section to compare the fabricated conditions with the specified connection detail. Statistical estimates of the as-fabricated weld geometry were determined from these measurements.

In this report, the rib-to-floor beam sections are identified by the specimen identification (FS for full-size specimen) suffixed by the side of the connection (e.g., N for North or S for South) and then suffixed by the location of the section measured in degree from the rib soffit at the centerline (e.g., 25 and 50). For identification of the sections at the intersection with rib-to-deck plate weld, section location is replaced by “C” which is abbreviation for corner. Total 6 cross sections were prepared with 3 sections on each side of the connection. The sections are shown in Figure 447.

The rib-to-floor beam weld sections were etched at with 5% Nital. The macro-etched sections were then digitally photographed. The dimensional parameters of the rib-to-floor beam weld were measured from the digital photographs of the macro-etched sections as described in the following. The photographs were imported into AutoCAD 2014 (Autodesk, Inc.). The nominal floor beam thickness was considered as the basis for the measurement. A scaling factor was computed as the ratio of the nominal floor beam thickness to the floor beam thickness measured from the digital photographs in AutoCAD. The other measured dimensional parameters were multiplied with this scaling factor to obtain the respective sizes. The dimensional parameters are shown in Figure 448. The measured dimensional parameters were: (a) the lack of fusion of the weld, *LOF* and (b) the root opening or fit-up gap between the rib and the floor beam, *R*.

The macro-etched sections of the rib-to-floor beam welds are shown in Figures 449 to 454. The destructive weld measurements on the macro-etched sections are tabulated in Table 44. The value of R ranged between 0.012 in. and 0.18 in. (~$^3/_{16}$ in.), with a mean of 0.068 in. (~$^1/_{16}$ in.) and a large COV of 94.5%. Such large COV can be expected considering the small magnitude of R. Out of 6 sections, the R value in 4 sections were less than the specified maximum of $^1/_{16}$ in. Large values of R were noted at sections FS_N_C and FS_S_C. These sections were at the intersection with the rib-to-deck plate weld. The LOF showed a much lesser variance (COV of 18.8%), with a mean of 0.374 in., or mean fusion of about 25%. Such small fusion or large LOF is typical of fillet welded connections. The weld profiles were typical of manual fillet welds with large LOF. At the rib-to-deck plate and the rib-to-floor beam intersections, however, the weld profiles were questionable.

Table 44 - Measured dimensional parameters of Rib 1-to-inner floor beam weld in full-size specimen

		Destructive Measurements by Lehigh	
SL. No.	ID[a]	LOF[b] (in.)	R[b] (in.)
1	FS_N_25	0.379	0.030
2	FS_N_50	0.478	0.022
3	FS_N_C	0.357	0.107
4	FS_S_25	0.302	0.058
5	FS_S_50	0.429	0.012
6	FS_S_C	0.300	0.180
	Maximum	0.478	0.180
	Minimum	0.300	0.012
	Mean	0.374	0.068
	Std. Dev.	0.070	0.065
	COV (%)	18.8	94.5

[a] Refer to Figure 468 for identification of sections.

[b] Refer Figure 448 for identification of weld parameters.

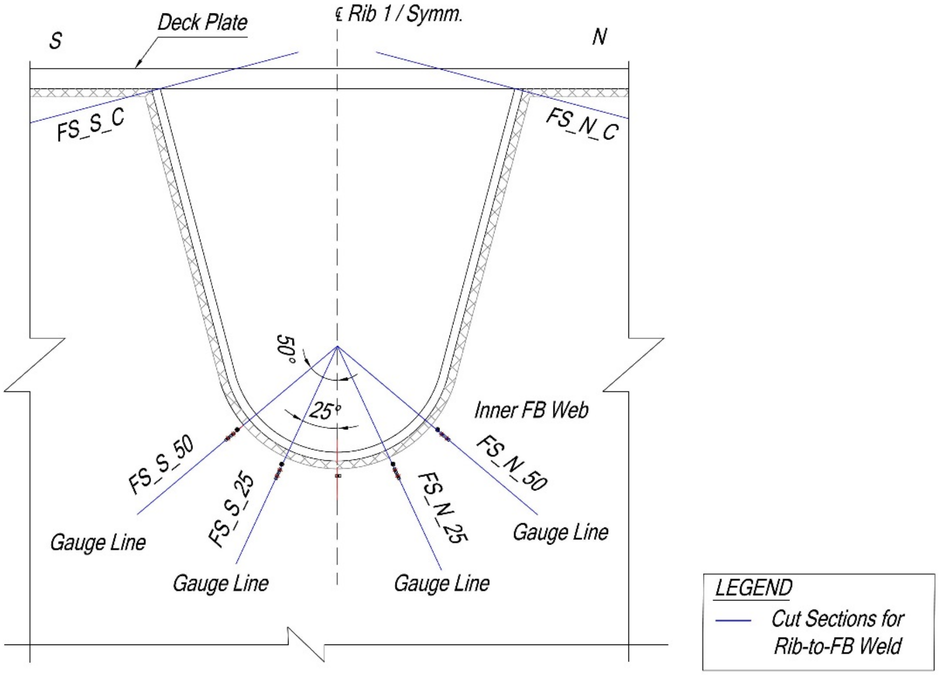

Figure 447. Section identification of Rib 1-to-inner floor beam weld

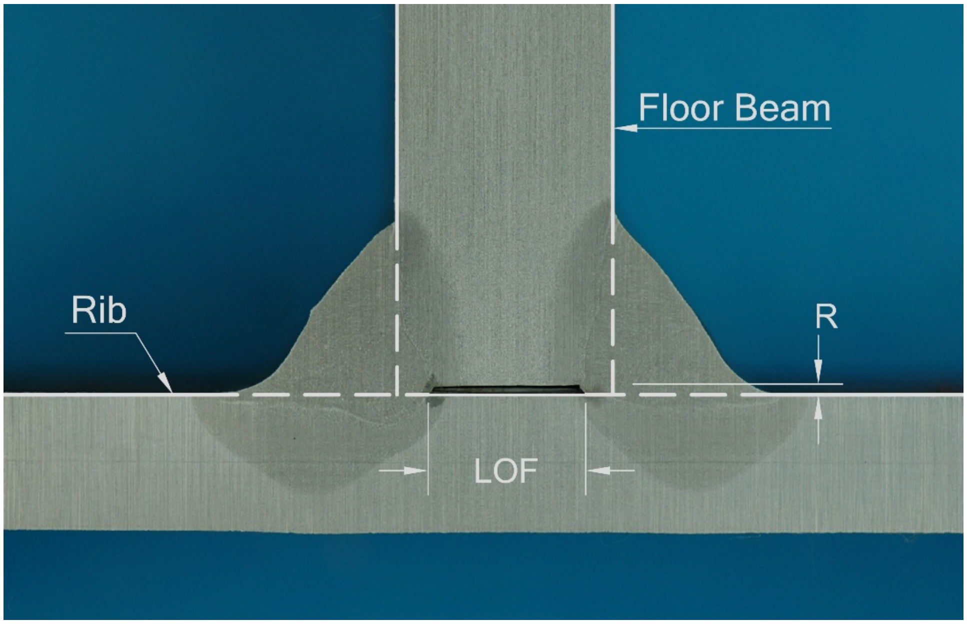

Figure 448. Schematic of rib-to-floor beam weld

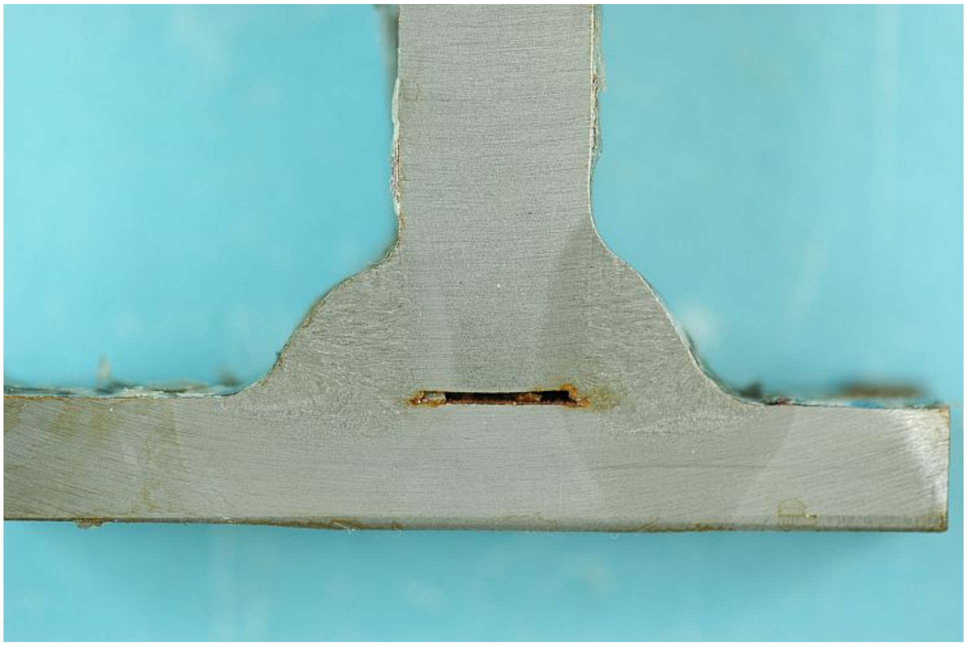

Figure 449. Macro-etched section FS_N_25

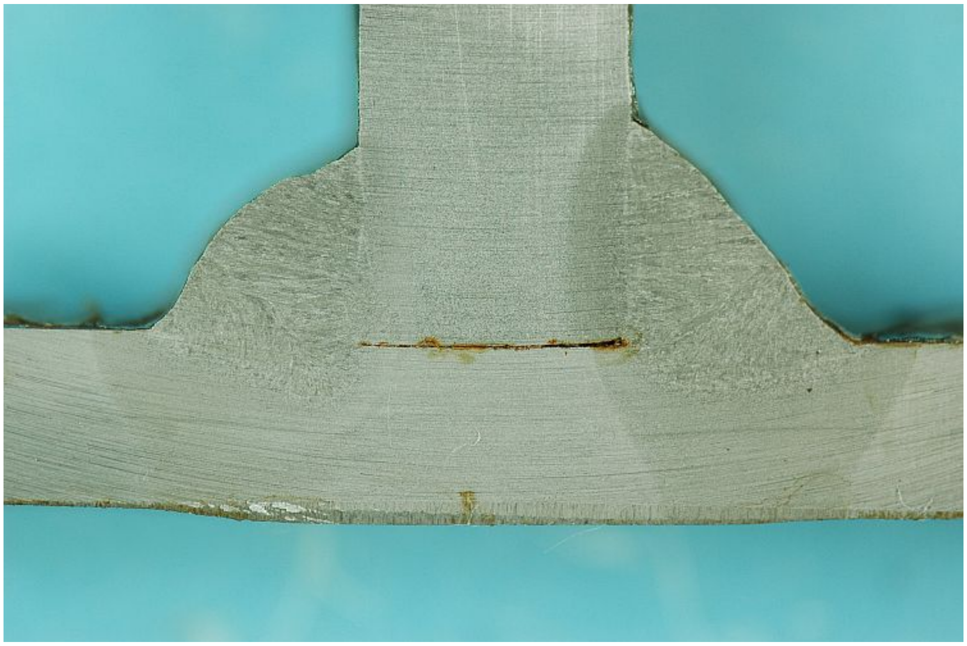

Figure 450. Macro-etched section FS_N_50

Figure 451. Macro-etched section FS_N_C

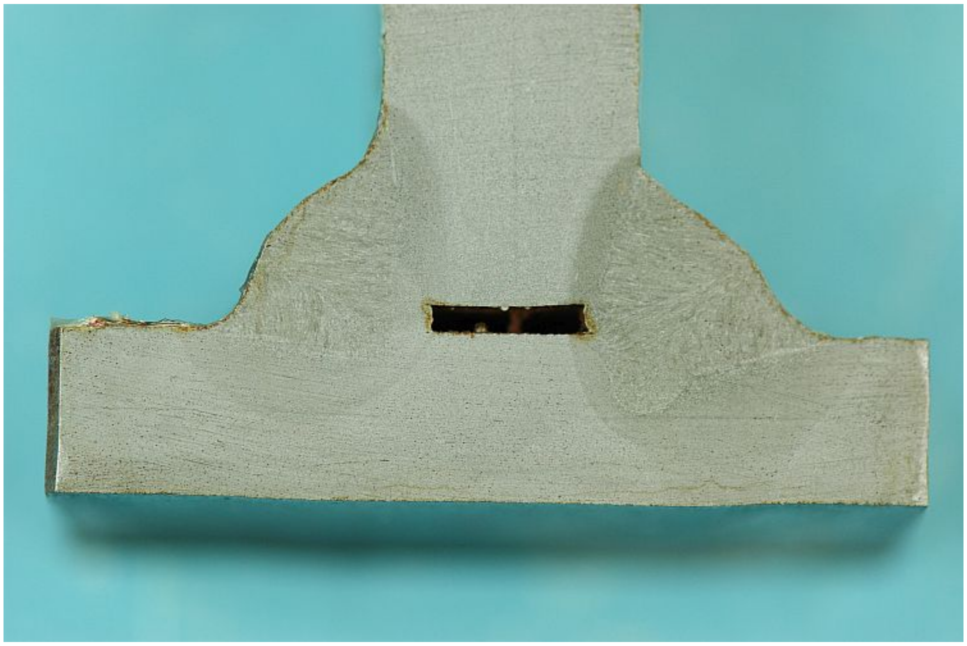

Figure 452. Macro-etched section FS_S_25

Figure 453. Macro-etched section FS_S_50

Figure 454. Macro-etched section FS_S_C

11. CONCLUSIONS AND RECOMMENDATIONS

11.1 Conclusions

The research project evaluated the design and fabrication of the proposed orthotropic deck for the Wittpenn Bridge replacement, for ensuring effective life cycle cost, including cost-effective fabrication and infinite fatigue life, i.e., no in-service fatigue cracking during the design life of the bridge. Contrary to the conventional design of modern orthotropic decks having $^5/_8$ in. (16 mm) deck plates, trapezoidal ribs and additional cutouts in the floor beams under the rib soffit, the proposed design incorporated a $^3/_4$ in. (19 mm) deck plate stiffened by rounded bottom ribs (U-shaped) passing continuously through matching cutouts in the floor beams without any additional cut-out under the rib soffit. The project objectives were accomplished primarily in two phases involving detailed FEA of the deck, evaluation of full scale small and large fabrication mockups consisting of different weld details, fatigue testing of small size mockups and full size deck prototype, and post-test destructive evaluation.

Multi-levels of 3D linear elastic finite element analyses (FEA) of the proposed bridge deck identified the rib-to-floor beam connection adjacent to a box girder as the most critically stressed region of the deck, when the rear tandem axle of the AASHTO fatigue design truck was symmetric with respect to the floor beam, and the rib was located in the shear span of the floor beam with the wheels nearest to the box girder placed centrally between the adjacent pair of ribs. The response of the orthotropic deck in general was characterized by global longitudinal and transverse flexure of the entire deck, and significant local deformations under the wheel loads. The stress concentration in the floor beam developed at the rib connection as the diagonal principal stress field in the shear span was interrupted by the rib opening. Stress concentrations were also noted in the rib wall around the rounding due to out-of-plane deformation resulting from the connection involving round geometry.

Three variations of rib-to-floor beam and rib-to-deck plate connection details, including the influence of different fabrication parameters, were explored in full scale small size mockups consisting of a single rib and one floor beam. Fatigue testing of these mockups in a novel test setup that simulated the stress distribution in the floor beam identified a unique failure mode of the rib-to-floor beam connection, where cracks initiating from the unfused weld root propagated into the floor beam web and the rib wall normal to the principal stress field, particularly when the stresses were significantly large. This study found that: (a) rib-to-deck plate connections employing a 70% PJP weld with a $^5/_{32}$ in. (4 mm) root face on the rib wall and maximum 0.02 in. (0.5 mm) fit-up gap; and (b) rib-to-floor beam connections employing a PJP weld with $^1/_8$ in. (3 mm) double bevel on the floor beam web and a maximum fit-up gap of $^1/_8$ in. (3 mm), were the most cost-effective.

Based on the FEA, a full-scale prototype of the part bridge deck comprising 5 ribs and 3 floor beams, and a test setup that would adequately replicate the boundary conditions were decided for assessing the in-service fatigue performance of the proposed deck by testing in the laboratory under simulated conditions during the final phase of the project.

Per NJDOT's recommendation, the following details were considered for evaluation in the prototype deck: (a) rib-to-deck plate connections: 80% PJP weld with minimum 70% penetration, no joint preparation on the rib wall and a maximum fit-up gap of 0.020 in. (0.5 mm); and (b) rib-to-floor beam connections: $^{5}/_{16}$ in. (8 mm) fillet weld with fit-up gap not exceeding $^{1}/_{16}$ in. (1.5 mm).

The prototype deck fabricated in two panels to simulate a transverse field splice showed that the lack of fit between the panels at the splice due to the distortion from welding heat effects can significantly exceed the specified tolerances of $^{1}/_{16}$ in. (1.5 mm). The residual out-of-flatness between the mating deck plates even after force fitting the deck plate at the welded splice can be as large as $^{3}/_{16}$ in. (9.5 mm). In addition, the study demonstrated that due to limited access it was practically impossible to perform the transverse splice of the deck plate as specified, i.e., depositing the CJP weld from the top with a backing bar underneath the deck plate, removing the backing bar, back gouging the weld root and re-welding in an overhead position. Use of a brass backing bar for easy removal resulted in significant LOF at the weld root, which could not be effectively repaired due to the access restrictions. Consistent with the existing practices for orthotropic decks in the United States, the most effective way of performing this weld at the transverse deck splice is to use a left-in-place steel backing and a wider root opening, which can ensure good fusion at the weld root.

The fatigue testing of the prototype deck, performed under the simulated rear tandem axles of the AASHTO fatigue truck for orthotropic deck design and subjected to the Fatigue I limit state load range of the AASHTO LRFD Bridge Design specifications 6th edition, was run out at 8 million cycles, without any detectable fatigue cracking in the deck. The measured stress ranges at all critical connections were less than the CAFT of their respective detail categories. The test results suggested infinite life performance of the deck design, as long as the site specific overloads do not exceed the AASHTO Fatigue I limit state load more than 1 in 10000 occurrences. The test results also demonstrated that deviation from the specified fabrication tolerances and rejected welding procedures, which were noted during the specimen fabrication and installation, did not affect the fatigue resistance of the connection details. These results suggest that further research is needed to develop appropriate fabrication tolerances and welding procedures.

11.2 Recommendations

The research developed cost-effective details for fitted rib-to-floor beam connections, and rib-to-deck plate connections for orthotropic bridge decks. In addition, the study provided critical information on issues related to fabrication and installation of the orthotropic deck design for the proposed Wittpenn Bridge, and the expected performance of the orthotropic deck in service under fatigue limit state loading, ensuring effective life cycle cost. The research also highlighted the need for developing rational tolerances for economic domestic fabrication of orthotropic decks.

The preliminary studies of the mockup specimens showed that a rib-to-deck plate connection employing a 70% PJP weld with a $^{5}/_{32}$ in. (4 mm) root face on the rib wall and maximum 0.020 in. (0.5 mm) fit-up gap, and a rib-to-floor beam connection employing a PJP weld with $^1/_8$ in. (3 mm) double bevel on the floor beam web and a maximum fit-up gap of $^1/_8$ in. (3 mm), were most cost-effective for the proposed orthotropic deck design. These details should be implemented for the proposed orthotropic deck design to achieve economic fabrication.

The laboratory studies of the full scale prototype demonstrated that deviation from the specified fabrication tolerances, which were observed during the specimen installation, did not affect the performance of the deck. A rational assessment of fabrication and installation tolerances is necessary and is highly recommended for cost-effective domestic fabrication of the orthotropic deck.

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