ORTHOTROPIC STEEL DECK BRIDGES IN THE U.S.

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Abstract

This paper summarizes some of the recent developments in the U.S. on the subject of orthotropic steel deck (OSD) bridges. The Federal Highway Administration published a manual to provide up-to-date technical guidance on the proper design, construction and maintenance of OSDs for bridges and the AASHTO bridge design specifications have been greatly revised and expanded. The rib-to-deck weld continues to be an area of difficulty in OSD construction; due to competing desire for fabrication economy and fatigue longevity in the detail. Preliminary results from FHWA research on this detail are presented.

Background

Many of the world's notable major bridge structures utilize the orthotropic steel plate system as one of the basic structural building blocks for distribution of traffic loads in decks and for the stiffening of slender plate elements in compression. Examples include the new San Francisco Oakland Bay Bridge, Self-Anchored Suspension Span in California and the proposed Strait of Messina Bridge in Italy. Stiffened steel plates have been used for many years in a wide range of steel construction applications. They are particularly prevalent in the ship building industry and for hydraulic applications such as tanks, gates, and locks. The first orthotropic steel deck (OSD) bridge was developed by German engineers in the 1930's and the first such deck was constructed in 1936. In the United States, a similar system was built and often referred to as a "battle deck" because it was considered to be as strong as a battleship. In recent years in the United States, there has been focused interest in bridge design concepts that are modular, prefabricated, and rapidly constructible. To provide updated guidance and encourage the use of OSDs, the U.S. Federal Highway Administration (FHWA) published the *Manual for Design, Construction, and Maintenance of Orthotropic Steel Deck Bridges* (Connor, et al. 2012).

An OSD typically consists of a steel deck plate with welded stiffeners or ribs parallel to each other in the longitudinal direction. Transverse cross beams are typically used to support the ribs and provide stiffness in the transverse direction. The transverse cross beams typically serve as floor beams transferring the deck loads to the main structure. These floor beams are often integrated with the deck structure where the top flanges of the floor beams are often the deck plate itself. The stiffening ribs can be open shapes such as plates, inverted T-sections, angles, and channels or closed box-type ribs with different geometric shapes; trapezoidal closed ribs are

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the most common. Figure 1 gives an illustration of a typical trapezoidal closed-rib OSD panel. The first orthotropic steel deck with closed ribs was constructed in Germany in 1954. Compared to open stiffeners, the closed ribs have many advantages. First, closed ribs can transfer the traffic load much more efficiently in the transverse direction. As a result, closed ribs can have wider spacing than open ribs. This results in fewer ribs that results in lighter weight, and less welding compared to open rib systems. Second, closed ribs can provide much higher flexural and torsional rigidity in the longitudinal direction allowing longer spans between transverse elements. In other words, fewer cross-beams are required, which further reduces the deck self-weight and the number of welds associated with the cross-beams. Lastly, since single-sided welds are used to attach the closed ribs to the deck versus double-sided welds for open ribs, the number of rib-to-deck welds is reduced by half. However, the one-sided welds required for closed-ribs can cause quality control and inspection issues which can be costly.

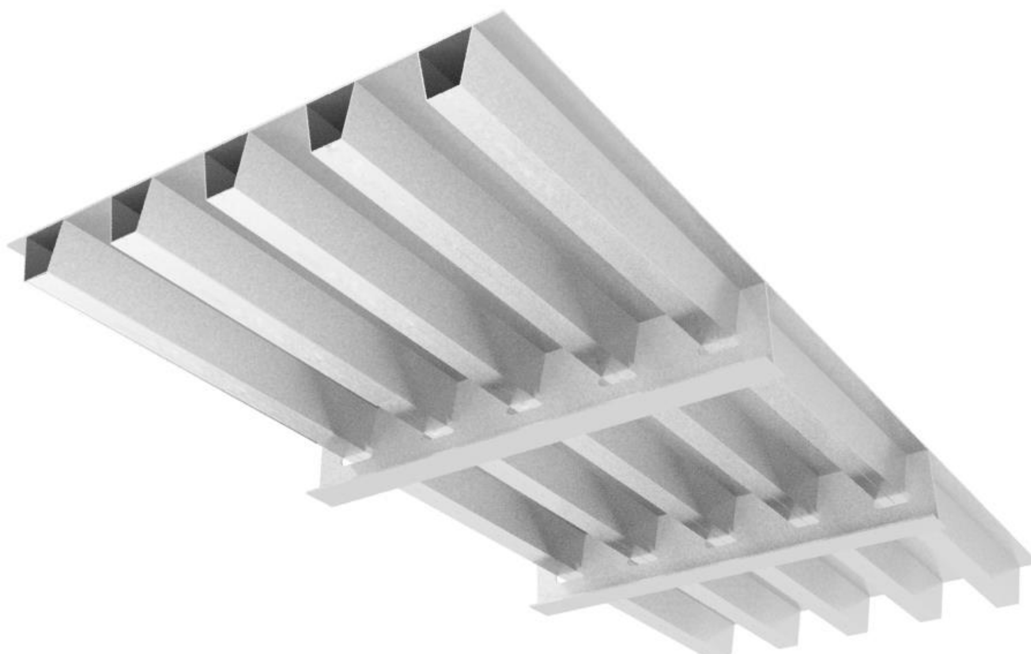

Figure 1. Typical closed-rib OSD panel

To overcome the challenges of one-sided welding and prevent premature fatigue failure, more careful consideration is needed to design rib-to-deck welds, and ongoing research is being done. Many of the earlier vintage orthotropic decks with closed ribs experienced fatigue cracking problems. There was a lack of knowledge about fatigue and a lack of guidance in the structural design codes. The complex stress state present at the rib-to-deck welds makes fatigue design even more difficult. The quest for lighter self-weight led to relatively thin deck plates in the structural design. However, many of the designs with thinner deck plates were vulnerable to high local stress effects from wheel loads. The contribution of the wheel-load effect was not fully considered in early deck designs and many bridges experienced fatigue cracking problems. Compared to main structural members, orthotropic steel decks tend to have a higher incidence of fatigue problems because of the local effects of wheel loads. Wheel loads cause large local stress variations, stress reversals, and an increased number of stress cycles that must be considered in fatigue design.

Design Methodology According to AASHTO LRFD

The applicable limit states for the design of OSDs include Strength, Service, and Fatigue, according the AASHTO LRFD Bridge Design Specifications (AASHTO 2014), here forth referred to as just “AASHTO”. All limit states need to be considered for complete design, but generally it is the Fatigue limit state that will control the majority of design details.

Strength

Strength design must consider the following demands: rib flexure and shear, floorbeam flexure and shear, and axial compression. The rib, including the effective portion of deck plate, must be evaluated for flexural and shear strength for its span between the floorbeams. The floorbeam, including the effective portion of the deck plate, must be evaluated for flexural and shear strength for its span between primary girders or webs. The reduction in floorbeam cross-section due to rib cutouts must be considered by checking flexure and shear where the portion of web is removed. When the panel is part of a primary girder flange, the panel must be evaluated for in-plane compressive strength based on stability considerations.

Testing has shown that OSD panels can have tremendous reserve strength for lateral loading beyond yield point due to membrane stiffening. This reserve, however, is dependent upon the boundary support conditions. For simplicity, the approach to strength design should conservatively limit stresses to the specified minimum yield strength or critical buckling stress.

Service

The Service limit state must be satisfied by limiting overall deflection for the deck plate to be less than the span length divided by 300, rib deflection less than their span length divided by 1000, and relative deflection between adjacent ribs less than 2.5mm (0.10 in.). These deflection limits are intended to prevent premature deterioration of the wearing surface.

Another applicable service limit state is the Service II limit state for the design of bolted connections against slip in the overload scenario. This should be considered for the design of bolted rib and floorbeam splices.

Fatigue

AASHTO introduces two fatigue limit states: Fatigue I for infinite-life design and Fatigue II for finite-life design. Due to fact that OSDs are governed by wheel loads (in particular the rib-to-deck connection), they experience multiple cycles of stress from every truck passage and thus will most often be designed for infinite life. However, finite life design may produce more cost-effective proportions when the traffic volume is not excessively high.

Design Load

For OSDs, it must be recognized that the AASHTO LRFD-specified 145 kN (32 kip) truck axle in the HL-93 load model actually represents a tandem consisting of two 71 kN (16 kip) axles spaced at 1220mm (4ft.). Thus, each wheel of the 71 kN (16 kip) axle is properly modeled in more detail as two closely spaced 45 kN (8 kip) wheels, 1220mm (4ft.) apart to accurately reflect an actual Class 9 tractor-trailer with tandem rear axles (see Figure 2). The single axle simplification is acceptable for main members not directly subjected to axle loads, such as girders, floorbeams, truss members, etc. However, research has shown that for elements directly loaded by wheels, such as expansion joints, the OSD plate, etc, this assumption is inappropriate.

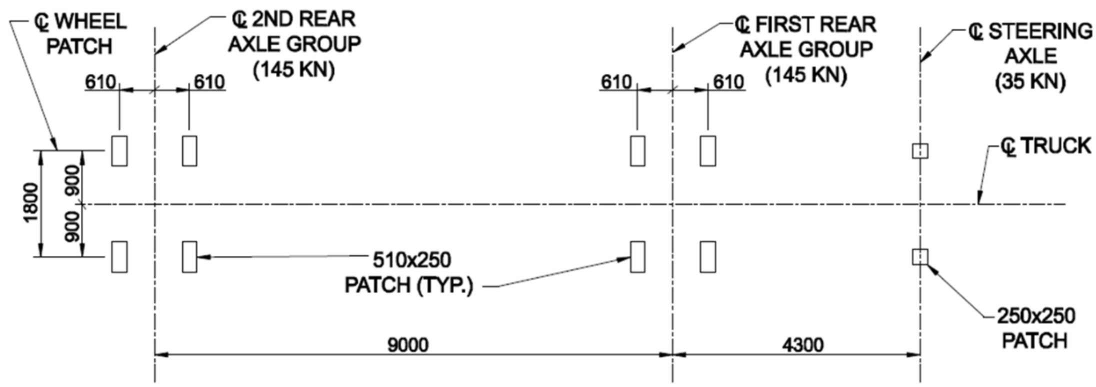

Figure 2 – Refined Fatigue Design Truck Footprint (all dimensions in mm)

Load Factors

For AASHTO fatigue design of OSD components and connections, the fatigue load factors are taken as 1.50 for Fatigue I and 0.75 for Fatigue II. There is an exception to this where it is increased to 2.25 for Fatigue I when checking connections to the deck plate and details around the floorbeam cut-out. The increased Fatigue I load factor is based on stress range spectra monitoring on both the Williamsburg Bridge and the Bronx Whitestone Bridge, which indicate that the standard Fatigue I load factor, which was developed for girders, floorbeams, truss members and other "global" components is unconservative for the design of certain OSD components. These studies indicate that the ratio of maximum stress range to effective stress range is approximately 3.0, which is larger compared to standard bridge girders. This is likely due to a number of factors such as occasional heavy wheels and reduced local load distribution that occurs in deck elements, as opposed to a main girder.

Cycles per Truck

The frequency of loading is critical for finite life design in OSDs. The Average Daily Truck Traffic (ADTT) and cycles per truck passage (n) both influence the total number of cycles for design. For components and connections of the OSD subjected to direct wheel loads, the number of cycles for design is governed by the number of axles expected to cross the bridge. Conversely, it is the number of truck crossings that equate to fatigue cycles for the main load-carrying members. For the refined tandem-axle truck, this results in five cycles per truck passage. However, it has been found that other components such as the rib and floorbeam typically experience only one primary stress cycle per truck passage. Thus, for design of all welded connections to the deck plate use n = 5.0 and for all others use n = 1.0.

Design Level

There are three acceptable Levels of analysis for design verification of OSDs that have been identified and are contained in the AASHTO specifications.

Level 1 Design

Level 1 Design is based on little or no structural analysis, but is done by selection of details that are verified to have adequate resistance by experimental testing (new or previous). When appropriate laboratory tests have been conducted for previous projects or on specimens similar in design and details to those proposed for a new project, the previous tests may be used as the basis for the design on the new project. All details must provide a level of safety consistent with the AASHTO LRFD specifications. Previously verified Level 1 designs may be used as the basis for design on new projects without additional testing, subject to approval by the Owner.

It is emphasized that the testing protocol must envelope the structural design loads and stresses for the new application. Test loading should be equivalent to the maximum truck load, and stress ranges at details should accurately simulate expected in service demands and should have accurate boundary conditions. For finite fatigue life design, the resistance shall provide 97.5 percent confidence of survival. For infinite fatigue life design, the constant amplitude fatigue limit (CAFL) should be exceeded no more than one in 10,000 cycles (0.01 percent). Full-scale test should include a minimum of 2 rib-spans with 3 floorbeams.

Level 2 Design

Level 2 design is based on simplified 1-D or 2-D analysis of certain panel details where such analysis is sufficiently accurate or for certain details that are similar to previous tested details as described in Level 1. Calculations consider only nominal stresses and not local stress concentrations. This is primarily intended to allow incremental improvement of previously tested details as verified by Level 1.

Details that are not subjected to local distortional mechanisms similar to those previously proven by appropriate laboratory testing or those that have been proven effective by Level 3 designs and long-term observation while subjected to the appropriate loads may be verified considering only nominal stresses with simplified analysis. Strength, Service, and Constructability limit states generally only require a Level 2 design.

Level 3 Design

Level 3 design is based on refined 3-D analysis of the panel to quantify the local stresses to the most accurate extent reasonably expected from a qualified design engineer. Level 3 designs will be dictated by the requirements to provide safety against fatigue failure. If no test data are available for a panel, Level 3 design is required unless it can be proven that the local distortional mechanisms (floorbeam distortion and rib distortion) will not lead to fatigue cracking. Level 3 is required for all bridge redecking applications unless the redecking procedure can be demonstrated as meeting the requirements of a Level 1 design and if approved by the owner.

Level 3 design is an extension of current AASHTO methodology for fatigue evaluation by nominal stresses. The proposed Level 3 design method is also a similar methodology applied by the American Petroleum Institute and American Welding Society and is well documented by the International Institute of Welding. It is used extensively for the fatigue evaluation of tubular structures and plate-type structures with complex geometries by various industries, where there is no clearly defined nominal stress due to complicated geometric effects. These are conditions very similar to orthotropic deck details. This approach recognizes that fatigue damage is caused by stress raisers that exist at details and attempts to quantify them by refined analysis rather than classification into general categories.

For fatigue design by such refined analysis, research has shown that the structural modeling techniques shall include:

- Use of shell or solid elements with acceptable formulation to accommodate steep stress gradients
- Mesh density of t x t, where t is the thickness of the plate component
- Local Structural Stresses shall be determined

FHWA's Rib to Deck Weld Research

Beginning in 2011, FHWA executed a multi-year program of research on the fatigue performance of OSD rib to deck welds. Fatigue design of the rib-to-deck weld is more complicated compared to the AASHTO fatigue design procedures for typical bridge members such as girders and crossframes. The AASHTO methodology relies on calculating a far-field (nominal) stress range that ignores any local stress concentration effects close to welded details. The stress ranges are caused by the overall truck loading event on the bridge. This contrasts with the rib-to-deck weld where local wheel load effects combine with the global truck event loads to create a complicated stress state. Because the closed rib-to-deck weld is typically a one-sided partial penetration weld, a crack-like condition remains at the root, which may initiate fatigue. Additionally, the weld toe is subjected to localized bending from the wheel load effects and cannot be reliably predicted with a far-field stress approach. Therefore, alternative approaches need to be established to analyze the fatigue resistance of the rib-to-deck weld. This section provides a brief overview of several alternative approaches that are currently available, with an emphasis on those chosen for study in this research.

Fatigue resistance is typically characterized into S-N curves where fatigue test data is plotted based on the stress range (S) and the number of cycles to failure (N) on a logarithmic scale. In logarithmic space, the fatigue data can be characterized by a straight line relationship for metallic materials. If the live load stress range is known at the detail, this relatively simple straight line relationship can be used to predict the number of cycles needed to cause fatigue failure. In fatigue assessment, the stress range values are calculated and used to predict the fatigue life, while in fatigue design the required number of service load cycles is used to determine the allowable stress range. The S-N curve is determined experimentally by performing logarithmic linear regression on test data developed at different stress ranges. Different welded details will have different S-N curves; therefore the curves need to be developed individually for each detail type. When fatigue resistance of a new type of structural detail needs to be evaluated, new fatigue tests need to be conducted to establish the S-N curve.

Alternative approaches have been developed where a locally calculated stress range is compared to a master S-N curve for the fatigue resistance of all weldments. This was investigated as part of this research, and will be referred to as the hot-spot stress approach.

To perform hot-spot S-N curve analysis, both the stress range local to the weld toe and the number of cycles to failure need to be obtained. The local weld toe stress range can be calculated using finite element modeling (FEM) of the welded structure. Because the local stress predicted by FEM is dependent on the element type and mesh density, the linear surface extrapolation (LSE) method is used. While this still requires special considerations for element type and mesh density, the results have essentially been normalized to one type of mesh size. Welds can be included or excluded in the modeling methodology.

The following section describes the experimental study and results from it. More details are forthcoming as the final report is still in review and is expected for publication in early 2015.

Experimental Study

A full-scale, small-specimen fatigue testing protocol was developed to investigate the fatigue resistance of this weldment. The stress range produced in actual orthotropic bridge decks due to the passage of vehicles is relatively complicated. It consists of both global bending stresses and localized bending and distortional stresses. Full scale testing of bridge decks under wheel loads would be the ideal way to study the rib-to-deck weld. However, such testing is cost prohibitive and cannot generate sufficient data to statistically study effects of various parameters. The full-scale, small-specimen tests are designed to accurately simulate the local transverse wheel load stress effect. These tests are a cost effective way to rapidly generate large amounts of data and study the significance of weld procedure variables. It is also a platform to evaluate the usefulness of the hot-spot method for fatigue design of this weldment.

The test specimens were fabricated by cutting 4.25 inch transverse strips from a full-scale rib-to-deck weldment (see Figure 3), then milled to an exact 4 inch width. The specimen is loaded in four-point bending with roller supports on the deck plate and load applied through the rib (see Figure 4). The application of force through the rib introduces stresses that simulate distortional effects in the rib and deck plate though the stress in the test specimens does not exactly correlate with the stress in real OSD under wheel loads. However, the specimens are still quite useful to understand the performance of this weldment under realistic conditions.

Fourteen panels were fabricated to explore variables such as welding processes (SAW, GMAW, and HLAW), six different weld penetrations, load ratio (ratio between minimum and maximum applied load), and root gap. This overall resulted in 185 individual specimens. The testing was performed both at the FHWA Turner-Fairbank Highway Research Center in McLean, Virginia and in the Thomas Murray Structures Laboratory at Virginia Tech University.

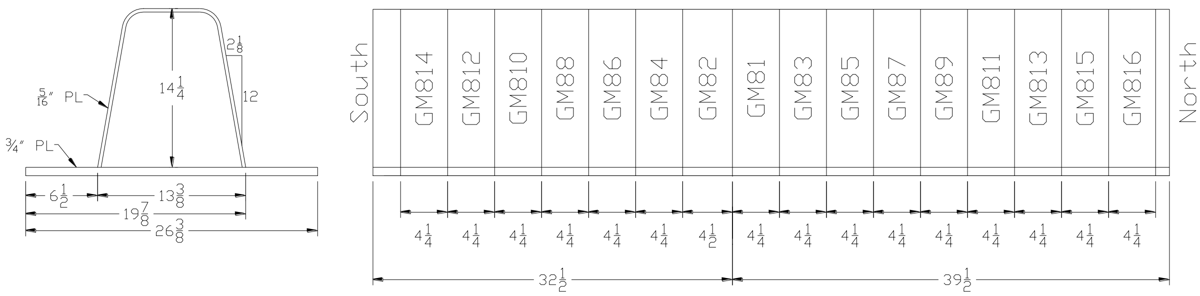

Figure 3 Example of full weldment, and cutting into smaller specimens.

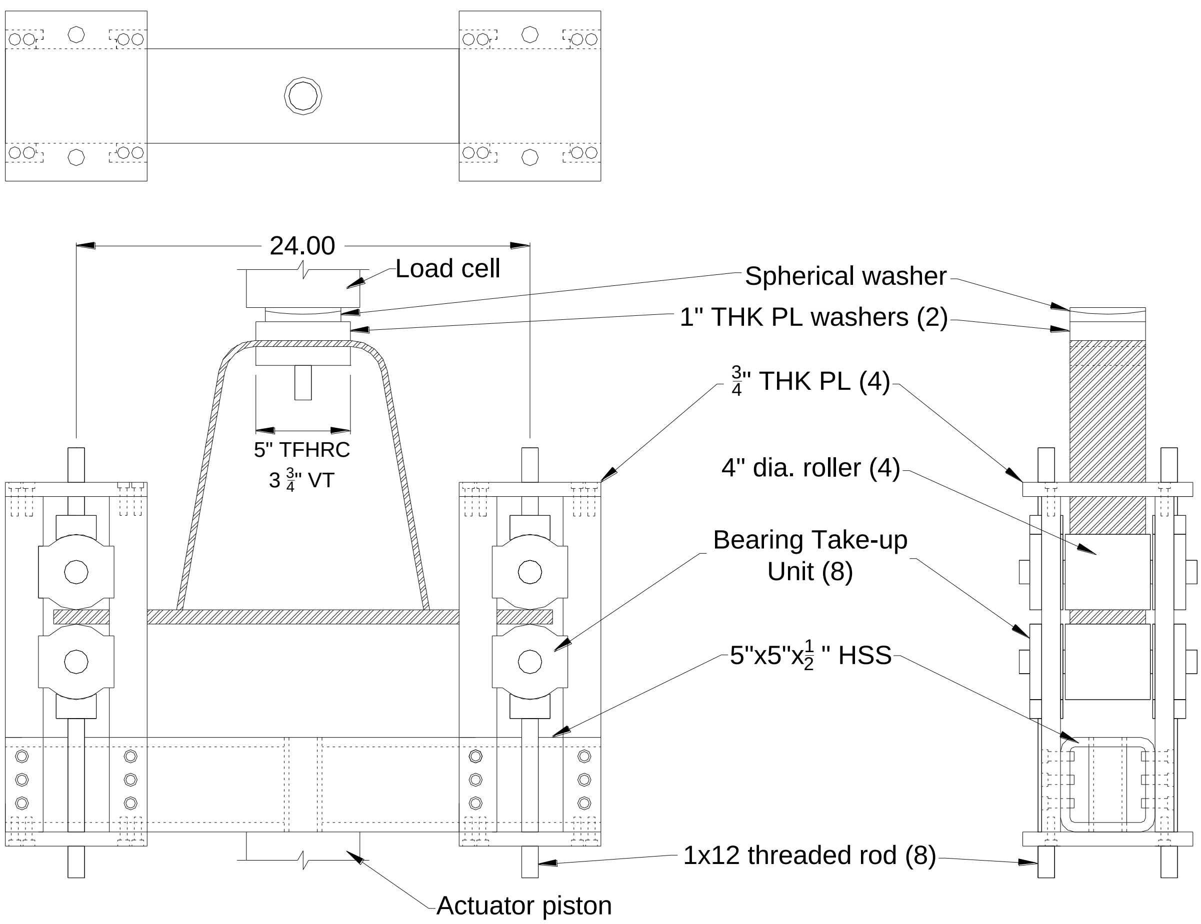

Figure 4. Test Setup.

Results

From the fatigue tests three failure modes were observed: fatigue cracks at the weld toe into the deck plate (WT@DECK), at the weld toe into the rib wall (WT@RIB) and from the weld root through the weld (WR). Testing performed at TFHRC mainly had WT@DECK failures, and the testing at VT obtained more WT@RIB and WR failure modes. Two reasons explain this difference, 1) TFHRC did not test specimens with intentional root gaps, which would increase the propensity of WR failures, and 2) the bearing plates used in VT were narrower (3.75 inches) than the ones used in TFHRC (5 inches), causing more rib distortional bending that slightly elevated the stress at the rib weld toe (see 04).

Solid element models of the specimens were used to calculate the local hot-spot stress at the weld toes for each specimen. If failure occurred at the weld toe, the hot-spot stress range was used to plot the data. For failures that originated at the weld root, there is no definition for the hot-spot stress range, and nominal stress range was used to report the data. The S-N data for many of the specimens is shown in 05. The data set have been combined based on the failure location (WR, WT@RIB, and WT@DECK) and the load ratio (R= -1 or 0). A load ratio of -1 implies complete load reversal, and zero is pure tension cycling. Also shown in this graph are the lower bound (mean minus two standard deviations) for each data set, and the AASHTO B, B', and C curves.

Prior to testing, all specimens had their welds etched and photographed. The digital photos were measured in a CAD program to determine the hypothesized critical dimension of each weld. These dimensions are identified in 0 and are defined as follows:

d_1 – leg size on deck plate
d_2 – penetration
d_2/d_4 – percent weld penetration
d_3 – length of the gap behind the weld
d_1+d_2 – total length along deck plate
h – leg size on rib
t – minimum throat size
A_w – total area of weld nugget

Once the weld dimensions were obtained, multiple linear regression was performed to determine the effect they had on the fatigue resistance. The result of this is shown in 07 as a Venn diagram where the circle entitled "Response" represents the fatigue strength. The other circles represent the weld dimensions that had a statistical significance on the fatigue strength, where circles with larger overlap are more influential on the fatigue strength. The three circles that overlap the most are the two leg dimension, and area of weld. The area of weld is proportional to the leg dimensions, so the finding was the rib-to-deck weld is primarily influenced by the size of the weld, not so much its penetration.

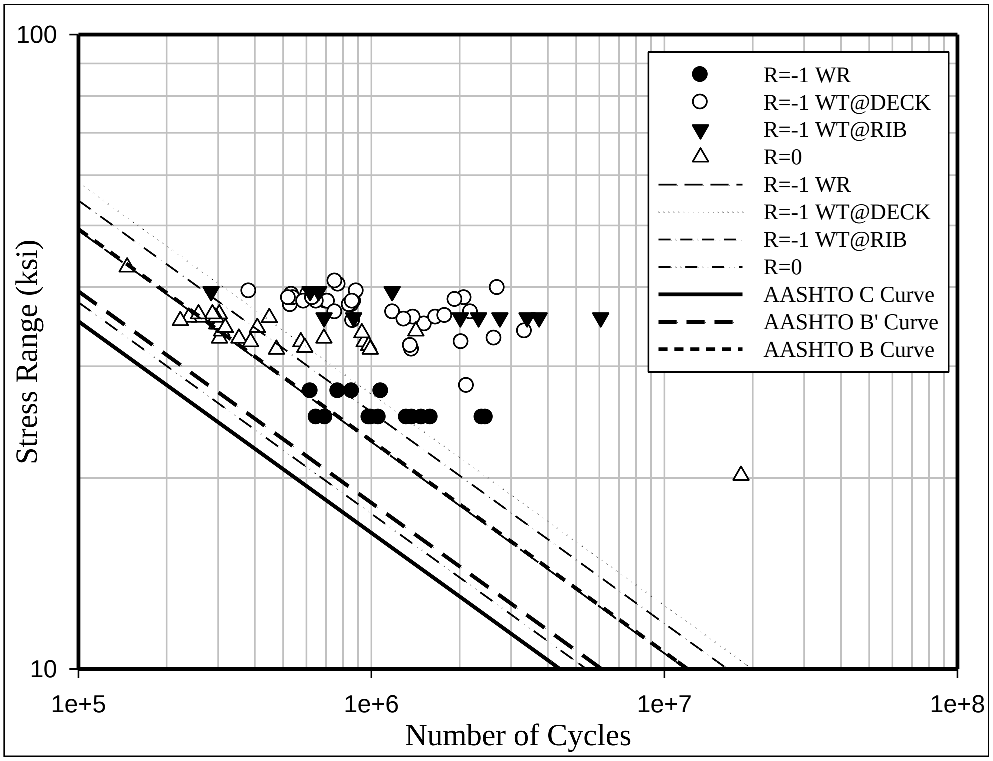

Figure 5. Lower bound S-N curves for all specimens.

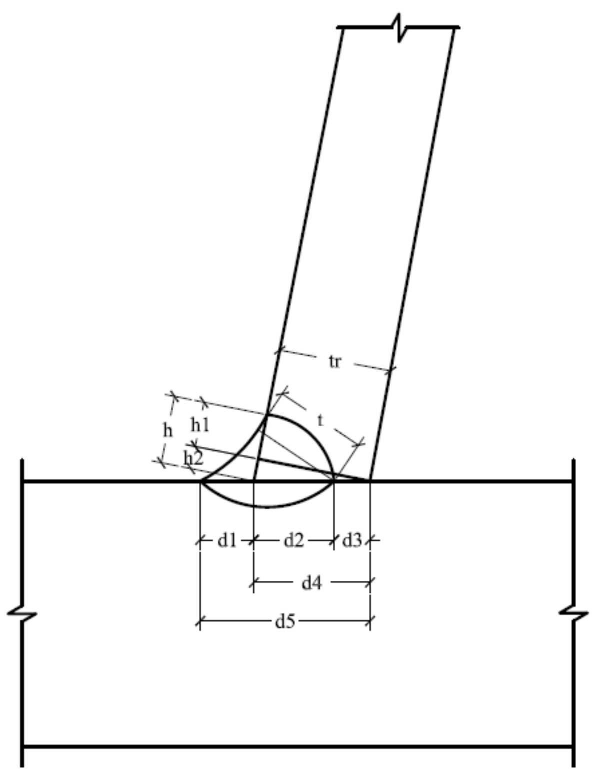

Figure 6. Measured dimensions of welds.

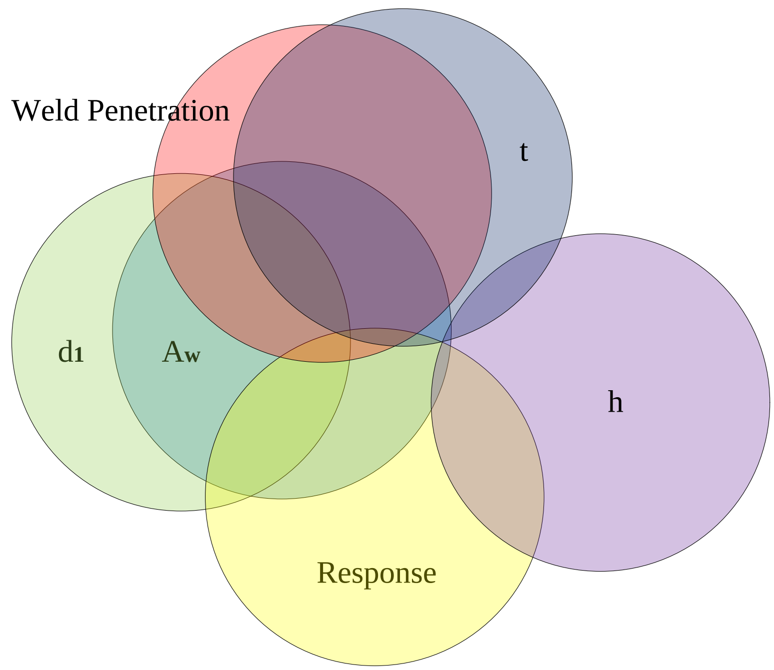

Figure 7. Interactions among weld dimensions on predicting the response (fatigue life).

Conclusions and Recommendations

1) The hot-spot fatigue analysis method was shown to provide good predictions of fatigue cracking initiating at the weld toes. For a lower bound, the fatigue life can be accurately predicted by comparing the hot-spot stress to AASHTO Category C.

2) The distance the weld toe extends beyond the plates was determined to have a significant effect on fatigue life. Viewed from the weld surface, this corresponds to the leg size of a fillet weld. Many of the welds tested had unequal legs. The fatigue resistance of the long leg weld toe increased while there was a corresponding decrease at the short leg weld toe. For a given weld area, altering the leg lengths had little overall effect since increased fatigue resistance at one weld toe was offset by decreased fatigue resistance at the other. In general, welds with relatively equal leg lengths are preferred. It was found that the leg size should be greater than 0.7 times the rib wall thickness to achieve AASHTO Category C fatigue resistance.

3) The weld area or weld size has a significant effect on fatigue resistance with larger welds providing better performance.

4) Weld penetration was found to have little effect on fatigue resistance as long as the root gap is closed after welding. In this case, fatigue cracks will occur at either of the two weld toes. The current requirements for 80% minimum penetration can be substantially relaxed if the root gap opening is controlled through proper fit-up and tacking of the plates prior to welding.

5) The openness of the root gap after welding is the significant parameter affecting fatigue cracking from the weld root. The root gap openness has little effect of fatigue cracks that initiate at the weld toes. The test results indicate that if the weld root gap is closed after welding than root cracking will not occur. There is a substantial benefit to designing the joint and controlling fit-up so that the root gap will close due to weld shrinkage.

6) The current 80% penetration requirement also appears to be an effective means of

preventing root cracking since the crack size is limited. Linear elastic fracture mechanics predicts that a smaller root crack size will reduce the stress intensity at the root crack tip. Reducing weld penetration without designing the weld for a closed root crack is not recommended since it will increase the probability of root cracking.

7) Holding a maximum gap tolerance of 0.020 in. during tack welding will result in a closed root gap after finish welding. Weld shrinkage is sufficient to close a 0.020 in gap as the weld solidifies and cools.

8) For specification requirements, it is recommended that the current 80% minimum penetration requirement can be reduced to 70% provided that the fit-up gap is less than or equal to 0.020 in. following tack welding. The data suggests that further penetration reductions may be possible under some conditions.

9) The recommended fatigue design philosophy for the rib-to-deck wild is to calculate the fatigue resistance at the weld toes using the hot-spot method based on FEA. FEA can capture the cumulative effect of stress from both local and global effects and can directly capture the effects of plate distortion. Root cracking should be prevented by controlling the weld geometry and fit-up tolerances of the joint.

References

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