DISCUSSIONS AND CLOSURES

Discussion of “Consistent Approach to Calculating Stresses for Fatigue Design of Welded Rib-to-Web Connections in Steel Orthotropic Bridge Decks” by Robert J. Connor and John W. Fisher


Roman Wolchuk, P.E., F.ASCE1
1Principal, Roman Wolchuk Consulting Engineers, 26 Journal Square, Jersey City, NJ. E-mail: Wolchuk@bellatlantic.net

The authors discuss fatigue strength of welded rib-to-web junctions at the deck supports with reference to redecking of the Williamsburg and the Bronx-Whitestone bridges, where low available depth necessitated the use of special details, and recommend the use of the “modified nominal stress approach” for the design of such connections. However, experience with thousands of existing orthotropic deck bridges throughout the world shows that applied live load stresses are not the only cause of fatigue cracking and that factors related to fabrication may be of equal, or possibly, greater significance. It has been reported that in many cases, cracks in orthotropic decks developed shortly after fabrication and welding, or early after the deck installation, before the live loads had been fully applied. This led to the conclusion that fatigue cracking occurs where the effects of heavy wheel loads coincide with faulty details or inappropriate fabrication procedures (PSP 2005). Therefore, consideration of the effects of live load stresses alone addresses only one aspect of the fatigue problem.

The authors imply that the type of weld used for the rib-to-web connection is accounted for by Detail Category assigned to the fillet, the partial and the full penetration welds. However, many other factors related to welding such as weld size (total heat input), welding method (manual/automatic), welding speed and sequence, weld imperfections, weld terminations, etc. may be more important. Dimensional fabrication tolerances and the degree of constraint of the plate elements to be welded are further factors that must be considered in the assessment of fatigue resistance. Tensile forces developing at welds due to weld shrinkage upon cooling could be a direct cause of cracking. These forces are directly related to loose fabrication tolerances and the geometric constraints resisting the shrinkage.

Welding (and other processes causing melting or plastification of steel, such as flame cutting, heat strengthening, cold pressing of ribs, etc.) causes metallurgical micro-structure changes of grain size and crystalline structure, which may cause embrittlement. Residual tensile and compressive stresses also strongly affect the structural behavior of the steel material.

Fabrication factors affecting fatigue resistance may be widely different for the various details of an orthotropic deck and depend on equipment and techniques used by the fabricators. These factors are unpredictable, may be different on various projects, and are generally unknowable to the designer. Design codes do not provide specific guidance for welding and fabrication tolerances of orthotropic decks and designers generally lack understanding of these problems, or consider them to be the contractors’ responsibility.

The crossbeam web “tooth” welded between two ribs (detail shown in authors’ Fig. 5) is an example of a highly restrained plate element. The interior bulkhead welded inside the rib in the plane of the crossbeam web does not permit any relaxation of tension caused by the rib-to-web welds and makes this detail highly vulnerable to fatigue cracking. Another disadvantage of the interior bulkhead is the difficulty of proper execution of the interior bulkhead welds and their “wrap-around” terminations inside the ribs. Such interior bulkheads, necessitating cruciform connections shown in Fig. 7, hurt, rather than help the fatigue strength of this detail (Wolchuk 2006).

As noted above, the complex and costly intersection details in this case (free cutouts at the rib bottoms requiring grinding, special welding procedures) were necessitated by the unusual geometric constraints, not encountered on typical orthotropic deck bridges. Therefore there is no need to use such details on orthotropic decks with adequate crossbeam depth (about twice the rib height), where much simpler and more economical rib-crossbeam intersections can be used (Wolchuk 2001).

Prevalent plastification of the base material in the heat-affected zones of the welds where fatigue cracks originate poses a question of applicability, at such locations, of the classic S-N fatigue theory based on linear elastic relationship between strains and stresses. How can we define the “stress range” in a material subject to yield stress? It should be noted that the Eurocode fatigue provisions (prEN 1993-1-9) (Eurocode 2004) state that the classic (Wöhler’s) S-N relationship is valid only if the total stress at the location considered is within the elastic range.

The complex mechanism of fatigue cracking of material beyond its elastic limit still remains to be clarified. A plausible mode of behavior of material in post-elastic range, subject to repeated application of actions causing alternating tensile and compressive strain cycles, with resulting (partly) elastic stress fluctuations has been suggested by Wolchuk (2006). Such interpretation, suggesting “strain range” as a relevant parameter, may possibly justify the use of the conventional S-N relationship as a means to roughly assess fatigue resistance. However, regardless of the approach used, the designer must have a good understanding of the actual physical properties and structural behavior of steel beyond the proportionality limit.

Verification of the calculated stresses at the critical details near the welds by strain gauge measurements on the structure is also unreliable. By definition, the “strain gauges” can only measure the superimposed strains caused by applied loading, which, in locations past the elastic limit, may no longer be proportional to the stress increments and do not reflect the actual stress conditions that include residual stresses.

Such considerations also apply to the use of analytical methods based on linear elastic theory for the assessment of stress concentrations and “hot spots” at fully or partly plastified locations. However, as the authors rightly note, the use of inelastic methods of analysis, which are extremely cumbersome and costly, would be totally impractical.
Furthermore, such analysis would require the knowledge of locations and extents of the plasticized zones, which depend on the unpredictable welding and fabrication factors.

The authors’ comprehensive discussion of the various analytical fatigue design methods, the difficulties with their application in practical design, as well as the necessity to consider the effects of fabrication factors suggest that an empirical approach, based on accumulated experience and further testing of critical details, may be the most appropriate to assure safety against fatigue cracking of orthotropic decks.

It should be noted that similar empirical design provisions have long been in force for some structural components of bridges such as, open and filled steel gratings, precast, prestressed standard concrete girders, deck panels, etc. The American Association of State Highway and Transportation Officials (AASHTO) bridge design specifications permit the use of data provided by the manufacturers of such items, based on prior research, in lieu of detailed analysis by the designer.

The existing design provisions for orthotropic decks of the AASHTO LRFD Bridge Design Specifications already exempt some details not easily amenable to analysis from numerical fatigue investigations, provided the stipulated geometric requirements are satisfied. Empirical fatigue provisions of other bridge design codes are even more extensive. The Eurocode (prEN 1993-2: 2004) (Eurocode 2004) exempts some types of orthotropic deck bridges from numerical fatigue proof if the recommendations for details, welding, and fabrication tolerances are followed. The Japanese bridge design specifications (JRA 2002) provide standard details of trapezoidal ribs and their welds, which can be used without fatigue analysis.

The discusser suggests that the existing empirical fatigue design provisions of the AASHTO LRFD Bridge Design Specifications for the rib-to-deck junctions (Art. 9.8.3.7) be extended to include the rib-to-crossbeam web intersections and other fatigue critical details. For this purpose, economical designs for these details should be prepared, applicable to typical orthotropic decks without unfavorable geometric constraints. Recommendations based on comparative testing should also be made for most appropriate welding procedures and fabrication tolerances. Orthotropic decks satisfying the specified design and fabrication requirements would then be exempt from numerical fatigue analysis. This would encourage a wider use of orthotropic decks that are excellently suited to fill the current needs for rehabilitation of deteriorating bridges and their roadways.

The authors have a distinguished record of testing orthotropic decks, and their input in the proposed testing program would certainly be very helpful in its successful implementation.

Correction of an inaccuracy noticed in the paper: the Pelican-Esslinger design method was proposed in 1957, not in 1938 as stated on p. 518. This method, adapted to the American design practice, was reported by Wolchuk (2001).

References


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Sante Camo¹
¹Associate Principal, Weidlinger Associates, 375 Hudson Street, New York, NY 10014. E-mail: camo@wai.com

The subject of fatigue of orthotropic decks has long awaited serious discussion on how to address fatigue details specific to this type of structure. Also, the multiaxial effects, which may be non proportional (i.e. orthogonal effects when not in time phase are non proportional—they do not maximize at the same time—and present problems in the estimation of stress range and number of cycles) have not been carefully considered for their impact on fatigue damage and life duration.

The authors discuss fatigue damage only in the plane of the plate element of interest. They mention three analytical approaches for the evaluation of details:

1. The nominal stress approach;
2. The hot spot stress approach; and
3. The fracture mechanics approach.

The use of fracture mechanics approach in design of bridges has not been standardized. It is complex and not easily adaptable to conventional engineering practice. The author’s waiver of this approach must be shared by thoughtful practitioners. The uncertainty of the residual stress and the fact that the propagating crack engenders, in some cases, complicated stress fields ahead of the crack are major stumbling blocks for assessing life. However, this approach undoubtedly helps identify significant damage mechanisms in the preparation of laboratory test specimens and in developing idealized and simplified life estimation of the detail.

The nominal stress approach is the one preferred by the practicing engineer. It is probably the only approach that the engineer is familiar with, as it is the standard method of assessing fatigue adopted by the American Association of State Highway and Transportation Officials (AASHTO) Code. It is therefore important that other methods be similar to this one to keep the analysis simple.

The authors discuss dealing with some details, which have no similitude with the AASHTO Code details. They propose the application of the hot spot stress approach to base plate surface and weld toe details. They also propose a change of the AASHTO formula for modifying the category of symmetrically loaded “T” joints when they are joined by partial joint penetration welds, instead of fillet welds.

The modification of the “T” joint fatigue category by the use of the formula developed by Frank and Fisher (1979) is a welcome change that can be made to the AASHTO Code and gives
the engineer much greater latitude than the presently specified detail as illustrated in Fig. 1.

It is noted that the AASHTO formula is the same as the proposed Frank-Fisher formula when it is modified to make $2a = t_p$. This newly proposed formula allows the engineer to proportion the PJP weld, especially in orthotropic decks, to have acceptable reduction of the Category C of the CJP weld, far better than could be done by increasing the size of the fillet weld alone by the current AASHTO formula.

Regarding the hot spot stress approach, the authors propose it for (1) the diaphragm plate (called base metal) wherein lies a sharply curved cut; and (2) stress concentrations at toe of welds. This approach recognizes that a singular stress range could be used for any detail, if the real stress at the hottest spot in the detail were accurately known, instead of the nominal stress. The stress range should be associated with similar material properties to those being investigated, and have the same or higher residual stress, to be conservative.

1. Base metal—To address the residual stress issue in a base metal condition, the authors propose the use of Category A resistance. The actual concentrated stress (as in the curved cut of a diaphragm) can be easily achieved by finite element analysis, provided the analytical model is convergent. Such detail is less likely to be affected by fatigue than the hot spot at the toe of the weld, where the residual stress is presumably much higher.

The proposed category makes sense because the residual stress at the cut-out would usually be less than that at the edge of a rolled plate. The roughness of the cut must be controlled, as the authors indicate.

2. Toe of weld—The authors state, on p. 523 that, “the stresses closer to the edge of the cut-out and closer to the weld toe are about two times greater than the location of the (strain) gauge” (as indicated by the finite element model). The authors show a model in which the finite element next to the rib stem is horizontal and without a wrap-around, the strain gauge is parallel to the cut out. It is not clear whether they were reading, the principal stress or the stress normal to the toe from the model. In the case of the deck they were testing, it would not make much difference because the cut-out is such that the edge of the opening is nearly perpendicular to the edge of the rib. Fig. 2 illustrates photographically the detail being discussed. The photo shows that the crack initiated at the toe of the weld and the point of highest curvature of the cut extends in a direction nearly perpendicular to that of the principal stress, not along the toe.

The authors also state that the stress concentrations for Category C are known to vary roughly from 2.5 to 3. Clearly, a minimum threshold of hot spot stress for this detail ($2.5 \times 68.9 = 172$ MPa) can only explain that the hot spot stress had to be greater than $2 \times 68.9 = 138$ MPa to fail, and that the principal stress was not the stress measured.

**Fig. 1.** Modification factor formulae for details similar to AASHTO Category C resistance detail

$$\begin{align*}
(\Delta F)_n &= (\Delta F)^*_n \\
&= \left(0.06 + 0.79 \frac{H}{t_p} \right) \left(1.1 \cdot \frac{t_p^{1/6}}{t_p} \right)
\end{align*}$$

**AASHTO Modification Formula**

$$\begin{align*}
(\Delta F)_n &= (\Delta F)^*_n \\
&= \left(0.71 - 0.65 \frac{2a_i}{t_p} + 0.79 \frac{H}{t_p} \right) \left(1.1 \cdot \frac{t_p^{1/6}}{t_p} \right)
\end{align*}$$

**Proposed Formula (Frank - Fisher)**

**Fig. 2.** Crack propagation from point of highest principal stress in Williamsburg Bridge deck testing

**Fig. 3.** Cracks in deck plate at the rib-diaphragm-deck plate intersection
Since the material at the toe (in the heat affected zone) has the same defects as the base metal and fewer than in the nugget of a weld, it is not far-fetched to conclude that the threshold resistance of the hot spot (i.e., at the stress concentration) should be closer to Category A, barin higher residual stresses there than in the base metal. Therefore, the hot spot threshold stress range should be 165 MPa (between a Category B and a Category A) if the principal stress is used.

The authors suggest, on p. 523 that the stresses should be measured or calculated away (at a proposed standard distance) from the toe and the result compared to Category C, as in the nominal stress approach.

It appears to the discusser that the benefits of the hot spot stress approach are thus waived. An accurate estimation of the principal stress at the toe could be the fatigue resistance for the number of cycles experienced. This, not being sufficient statistically to determine a hot spot stress category for toe conditions could be compared to a multitude of data of Category C specimens from which the actual maximum principal stress could be extracted, using finite element of the detail tested. This will permit the establishment of a hot spot fatigue category, based on maximum residual stress. Or could otherwise be called a Category B (probably conservatively), similar to ground flush weld material.

Once a hot spot category for weld toes is derived from this data, it could be used at the toe/cut-out region, regardless of the direction of the cut relative to the rib or direction of principal stress. The principal stress, as determined from finite element, is with this new category. Then, there would be no need for prototype testing. This notion presumes nonpreferential direction of failure (i.e., failure will occur in the direction normal to principal stress, regardless of grain structure or defects which are presumed equally random in any direction). This tool is important because different practitioners have used different cut-outs on a "copy from another project" basis, and what looks good, but without sufficient supporting information.

The need to develop fatigue criteria to address unusual details poses a question for our industry—The work of backing into such fatigue category from available data needs to be done to advance this more powerful approach and standardization of the finite element analysis must accompany this effort. Who will do it and at whose expense? The practitioner cannot be burdened with this process while developing contract drawings for a project.

The authors, aside from the cruciform weld, did not mention the more generalized Notch Stress approach, which is discussed in fatigue literature (Radaj and Sonsino 1998). This approach is applicable to many varieties of weld root notches for which an equivalent stress concentration factor is applied. The approach uses the finite element method to obtain the stress concentration around an arbitrary notch, which must then be calibrated by testing. Presently, there is insufficient data, relative to resistance. Each detail, with its own residual stresses, needs to be tested separately. This approach may be developed further, once we have the necessary data.

Fig. 3 shows crack propagation at the deck plate where it meets the rib and the diaphragm—rib-diaphragm-deck plate detail (RDDP). The darker crack line (lower) in Fig. 3 is the best recollection of what the discusser saw at the Lehigh tests of the Bronx—Whitestone prototype. The lighter line is closer to what was presented photographically by Miki (2006).

There are many experiences in Japan of this type of failure and Prof. Miki used the Notch Stress approach to evaluate resistance. Attempts to contact him regarding the results of his investigation, and assumptions he used, were unsuccessful.

For this detail, the Notch Stress approach may be valid for determining threshold stress range (presumed to be very low) with the use of data that includes residual stress. However the life of such detail can only be determined either experimentally, or by a fracture mechanics approach, which is even more complex and out of the reach of the practitioner. Experimental work can be carried out with fidelity to nonproportionality of multiaxial effects and can yield far better information than a fracture mechanics approach, which requires many assumptions.

This knowledge is essential for establishing rules of design in which the use, or not, of the bulkhead plays a central economic role and is also dependent on the rigidity of the floor beam.

In recent years there have been several major bridges newly designed with orthotropic decks or those whose existing decks have been replaced with orthotropic decks. The value of this work amounts to roughly two billion dollars, including the newly designed San Francisco-Oakland Bay Bridge. Orthotropic decks are also essential for cable-stayed bridges and Bascule bridges. Yet, the industry is not capable of determining the serviceable life of these structures unless full scale prototypes are tested.

Following the Sacramento Conference in 2002, divergent opinions regarding the need to test emerged. Some renowned practitioners have become the champions of the No-Laboratory-Research attitude, and have excluded finite element analysis, relegating the orthotropic deck design practice to the idealizations of the 1950's, with lack of trust in fatigue science. The argumentation for this position is that the residual stress is at yield and any live load causes plastic deformation. But laboratory specimen, from which the fatigue data of the AASHTO Code was derived, also had residual stress of typical fabrications. Such arguments, therefore, not only demonstrate lack of knowledge of a subject fundamental to orthotropic decks, but do a great deal of harm to the advancement of these structures, which could indeed provide great benefits with improved knowledge.

It is noted that cracks in diaphragms are not catastrophic, but they may become ubiquitous to the structure and a nuisance to maintain. The need to test is essential to avoid this fate and propel these structures in the deck replacement industry, where they could prove advantageous.

References


Discussion of “Damage Localization and Finite-Element Model Updating Using Multiresponse NDT Data” by Masoud Sanayei, Erin Santini Bell, Chitra N. Javdekar, Jennifer L. Edelmann, and Eugene Slavsky

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Q. Qin
Dept. of Civil Engineering, Tsinghua Univ., Beijing, China 100084.
E-mail: qinqq@tsinghua.edu.cn

Discussion points on the original paper follow.

1. Multiresponse parameter estimation requires measurement of the same subset of multi degree of freedom (MDOF) (p. 693, line 15 from the bottom), and includes both static and vibration measurements. However, types of sensors for static and vibration testing are different and optimum localization of sensors requires different locations for different sensors. The requirement to measure the same subset of MDOF’s is not effective.

2. The analytical model of the UCII grid has 21 DOF’s if lumped masses are used and only vertical displacements are considered. Therefore, there are 21 modes. Higher modes are more sensitive on local damage. Why did the authors consider only the first 9 modes?

3. In Eq. (16), what are \( N_{ij}, P_{ij}, P_{ik}, \) NMDOF, NSF, NUP? There are no definitions of them in the paper.

4. Actually, bridge damage does not change element masses. For instance, cracking never change masses, and corrosion changes masses of some surface areas of steel parts of structural components but never changes imposed dead loads and live loads. In fact, even a very deep singular cracking does not change the axial stiffness of a truss bar. The authors’ opinion “even small differences in assumed material properties can affect the mass and dynamic responses such as natural frequencies and mode shapes of the structure” is doubtful.

5. In Table 3, all 21 vertical displacements (MDOF) are used for the error function MF. Did the authors try some sparse MDOF for this?

Closure to “Damage Localization and Finite-Element Model Updating Using Multiresponse NDT Data” by Masoud Sanayei, Erin Santini Bell, Chitra N. Javdekar, Jennifer L. Edelmann, and Eugene Slavsky

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Masoud Sanayei1; Erin Santini Bell2; Chitra N. Javdekar3; and Jennifer L. Edelmann4
1Professor, Dept. of Civil and Environmental Engineering, Tufts Univ., Medford, MA 02155 (corresponding author). E-mail: masoud.sanayei@Tufts.edu
2Assistant Professor, Dept. of Civil Engineering, Univ. of New Hampshire, Durham, NH 03824. E-mail: erin.bell@unh.edu
3Prudent Solutions Inc., Somerville, MA 02144; formerly, Ph.D. Candidate, Tufts Univ., Medford, MA. E-mail: chitra.javdekar@gmail.com
4LeMessurier Consultants, Cambridge, MA 02139; formerly, Graduate Student, Dept. of Civil and Environmental Engineering, Tufts Univ., Medford, MA 02155. E-mail: jedelmann@lemessurier.com

Response 1: The writers wish to respond the discussers’ enumerated comments on page 695, Table 3 shows that 21 vertical measured coordinates of the five modes used. Paragraph 2 of the same page indicates that three static load cases were used with 9 (typographical error in the original paper, written as 11) measured displacements. These are two different sparse subsets of degrees of freedom (DOFs) available for measurements. Stacking of static and modal error functions, as shown in Eqs. (14) and (15), can be used for parameter estimation. Using stacking of error functions, it is not required to have the same subset of measurements in static load cases, modal measurements, or a combination thereof. More case studies with stacking of error functions are presented in Santini-Bell et al. (2007).

Response 2: The University of Cincinnati Infrastructure Institute was responsible for nondestructive testing and modal identification. A better match was found in the first nine modes as various mathematical techniques were used for modal identification. It seems identification of higher modes might be more challenging.

Response 3: The variables used in Eq. (16) are:

\[ E_{ij} = \text{error function at any interaction, cell } ij; \]
\[ E_{0ij} = \text{error function evaluated at initial values, cell } ij; \]
\[ E_{njij} = \text{normalized error function at any interaction, cell } ij; \]
\[ P_{ij} = P_{ik} = \text{error function at any interaction, cell } ij; \]
\[ \text{NMDOF} = \text{number of measured degrees of freedom}; \]
\[ \text{NSF} = \text{number of sets of forces (or frequencies)}; \]
\[ \text{NUP} = \text{number of unknown parameters}. \]

Response 4: The authors concur that bridge damage does not necessarily change structural component masses. However, discrepancies in stiffness and mass properties of the model can both change modal properties of a structure. In some cases, even without any damage to the structure, the initial values of mass and stiffness properties are simply inaccurate due to modeling errors. In these cases, it is essential to perform parameter estimation for model updating using both stiffness and mass properties of the structure.

Response 5: We used all 21 measured displacements that were available. This is indeed a sparse subset of all degrees available for this three-dimensional grid model. The authors have previously experimented with using various subsets of DOF for measurements as reported by Sanayei and Onipede (1991) and Sanayei et al. (1999).

References

