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FATIGUE STRENGTH OF PLATE GIRDER IN BENDING CONSIDERING OUT-OF-PLANE DEFORMATION OF WEB

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▶ Discussion ——— *By Chitoshi MIKI* (*Tokyo Institute of Technology*) *and Ben T. YEN* (*Lehigh Univ.*)

The authors are to be commended for their work in correlating in-plane and out-of-plane bending stress in plate girder webs. However, their evaluation of fatigue strength of plate girder webs due to such out-of-plane bending does not proceed in the appropriate direction, and the subsequent conclusions are erroneous and misleading.

The followings are a number of relevant points.

(1) The limits of the web slenderness ratio in AASHTO Specifications are based on the fatigue tests performed at Lehigh University in the 1960's. Type 1 fatigue cracks occured in very thin plate girders with high slenderness ratios (over 200), relatively large aspect ratio (1.0 or more) and large initial deflections ($w_0/t_w \geqq 1$)[1]. Until now, a type 1 crack has not been observed in actual bridge girders in Japan and U.S.A.. Considering the current Japanese specifications[21] and the values of structural parameters of bridge girders, such as the web thickness, slenderness ratio, aspect ratio and measured imperfection[22], the possibility of the occurrence of a type 1 crack may be very low.

(2) Actual restraint from the girder compression flange to the web plate out-of-plane bending depends on the torsional rigidity of the compression flange. Results of analysis and evaluation of the measured stresses from literature have shown that a fixed-edge condition over estimates actual stresses[2]. Mathematically predicting out-of-plane bending stresses for the purpose of assessing fatigue strength can not be expected to give reliable results.

(3) The concept of assuming a given fatigue life cycle and specifying the fatigue strength of a structure detail through the stress ratio, R, has not been providing satisfactory basis for fatigue strength evaluation for welded structural details. Stress range has been found to be the governing factor for such details[23),24)].

(4) Residual stresses along web boundary can influence web buckling stresses, as it is discussed in the paper. On the other hand, results of experiments in laboratories and examinations of numerous actual bridge structures[25] indicate that, for welded structure details such as plate girder web boundaries, the existence of residual stresses is one of the factors leading to the validity of the concept of stress ranges. To discuss the influence of residual stresses on fatigue strength through the consideration of web plate buckling is not valid.

(5) Test results have shown that for welded structural details, fatigue strengths are not influenced by the yield stresses of materials. To discuss the influence of yield stress on fatigue strength through the parameter of web buckling is also not valid.

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▶*Closure*

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The writers wish to thank Prof. Miki and Prof. Yen for their interesting and valuable discussion of the writers' paper.

(1) The limits of web slenderness ratio in the load and resistance factor design in the AASHTO Specification[6] are not directly based on the fatigue strength of fillet welds subjected to secondary bending stress. So, in the fatigue tests carried out by Toprac[4], Yen[1),3] and Maeda[5], type 1 fatigue cracks did not occur in some web panels of the web slenderness ratio above the limits. Inversely, even in the web panels below the limits, the fatigue cracks would have occurred if the secondary bending stress had exceeded the fatigue strength of fillet welds. The validity of the limits for the web slenderness ratio in the AASHTO Specification will be proved after the relation between in-plane bending stress and web slenderness ratio, which is directly based on the fatigue strength of fillet welds subjected to secondary bending stress, has been obtained.

The writers[10),14] have already made it clear from comparison of the experimental results of Toprac, Yen and Maeda with their analytical ones that not only the magnitude of the initial deflections, but also their shape greatly influences the initiation of type 1 fatigue cracks. When a web panel of aspect ratio 1.0 has an initial deflection of mode 2, as can be seen from Fig. 8, the 2×10^6 cycles fatigue strength given by in-plane bending stress is very different depending on the magnitude of the initial deflections. Accordingly, the fatigue cracks due to out-of-plane deformation of the web cannot be prevented by limiting the web slenderness ratio alone. Furthermore, some provisions should be added for the initial deflection shape.

At present, in the Japanese Specification for Steel Highway Bridges[21], the allowable stress design has been applied, and articles of fatigue design are not specified yet. As can be seen in the AASHTO Specification[6] adopting the load and resistance factor design or in BS 5400[26] adopting the limit state design, the tendency of bridge design in the world is moving from the allowable stress design to LRFD or LSD, and the use of more slender web is approved in those new design. The writers' paper will give a guidance to the establishment of serviceability in LRFD or LSD.

(2) Including the problem pointed by the discussers, the writers have already explained in Refs. (10, 14) the reasons why the estimated values of secondary bending stress is larger than the measured ones.

a) The measured values show the secondary bending stress at the point a few cm apart from the toe of a fillet weld, which is in general smaller than that at the toe.

b) The estimated values include only the effect of single-mode components of the initial deflection, but do not give combined effects of each mode. For example, in the case of a web panel of aspect ratio 1.0, the mode 1 component of the initial deflection has such an effect that the out-of-plane deflection and the secondary bending stress do not increase so much.

c) The boundary conditions do not fully agree with those around the web panel of actual plate girders.

Referring to c), the influence of torsional rigidity of the compression flange has already been revealed in

Ref. (10). In the plate girders used in the fatigue tests by Toprac, Yen and Maeda, as the web becomes thinner, the rotational restraint of the flange to the web becomes larger. The web panels of web slenderness ratio above 200 can be regarded as fixed-supported by the flange.

By the way, the boundary conditions in the in-plane direction around the web panel also influence the out-of-plane deflection and the secondary bending stress. The writers showed in Ref. (16) that an increase in the out-of-plane deflection of the web was very different depending on the boundary conditions in the in-plane direction. The values of coefficients shown in Table 1 are obtained for the case where the deformation in the in-plane direction around the web panel is not restrained by flanges, vertical stiffeners or neighboring web panels. The degree of the restraint in the in-plane direction around the web panels in actual plate girders will be given by structural analysis of the overall girder.

(3) The writers also know that a stress range is a governing factor of the fatigue behavior of welding joints. So, as can be seen from Eq. (23), the 2×10^6 cycles fatigue strength of fillet welds subjected to secondary bending stress is given by the stress range independently of the stress ratio. But, the 2×10^6 cycles fatigue strength given by the range of in-plane bending stress varies depending on the stress ratio, because the relation between in-plane bending stress and secondary bending stress obtained by Eq's. (6) and (11) is nonlinear. Then, the writers gave the 2×10^6 cycles fatigue strength with the maximum in-plane bending stress for each value of the stress ratio.

(4) The writers, too, admit the discussers' opinion that in welding joints, the existance of residual stresses due to welding operation is one of the factors leading to the validity of the concept of stress range. So, as has been mentioned in (3), the 2×10^6 cycles fatigue strength of fillet welds subjected to secondary bending stress is given by the stress range. But, the fatigue strength given by the in-plane bending stress decreases due to the residual stresses, because the buckling coefficient decreases. The fatigue strength given by the secondary bending stress is influenced by the factors related to the welding joints, but the fatigue strength given by the in-plane bending stress is influenced by the factors related to the structural behavior in addition to the above ones.

(5) The writers applied the condition of Eq. (23) to the 2×10^6 cycles fatigue strength of fillet welds subjected to secondary bending stress, regardless of the yielding stress. As has been mentioned in (4), the fatigue strength given by the in-plane bending stress is influenced by the factors related to welding joints and by the ones related to the structural behavior. As can be seen from Eq. (6), the buckling coefficient is one of the factors related to the structural behavior. Using the parameter of Eq. (27), the buckling strength can be expressed independently of the yielding stress. Then, the writers examined the relation between the 2×10^6 cycles fatigue strength given by the in-plane bending stress and the parameter.

APPENDIX——REFERENCE

26) BS 5400 Part 3 : Code of Practice for Design of Steel Bridges, British Standards Institution, 1982.

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