Fatigue durability evaluation of orthotropic steel bridge deck regarding fatigue crack in trough to deck plate detail

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1. Introduction

Fatigue cracks in deck plate of trough to deck plate details of standard orthotropic steel decks have been reported recently, Fig. 1. This kind of crack provokes concerns on performances of the steel decks as it occurs in a hidden location and reduces the bearing section of deck plate. Therefore, the first problem to solve is to evaluate the fatigue durability of the steel deck under a known traffic condition. Now, we will take a close look at a procedure for such an evaluation. The evaluation procedure consists of three steps: fatigue strength evaluation, stress range computation, and fatigue life estimation.

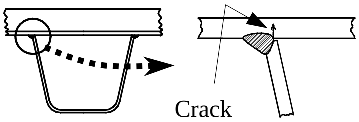

Fig 1: Trough to deck plate detail with a cracked in deck plate

2. Fatigue strength evaluation of trough to deck welded detail

Fatigue strength of the detail of standard orthotropic steel deck is predicted using a so-called "one-millimeter stress method" [1]. In order to apply this kind of geometric stress method, several factors are necessary; two of which are fatigue strength of reference detail, and correction factor that firstly dealing with computing stress at one mm from weld root in thickness direction by FEA, called one-mm stress. The reference detail is assumed to be a 10mm thick plate small cruciform joint. Its fatigue strength is obtained from fatigue tests on 160 and 200mm wide joints with axial loading. The model generated in FEA for computing one-mm stress is shown in Fig. 2. A point to note is that model is subject to bending stress only; that is bending stress governs fatigue life of the joint. An answer to this assumption is that bending stress is dominant in the deck plate according to results from a series of FEA on two-panel deck model using shell element with various loading cases as will seen in the coming section. It should be pointed out that the end of rib side is free from any constraints or forces as they have very small effects on the crack under discussion.

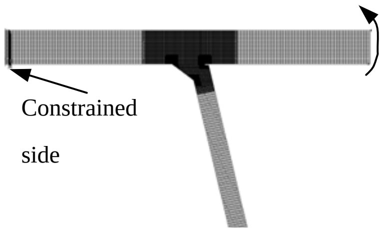

Fig 2: A FE model used for fatigue life prediction

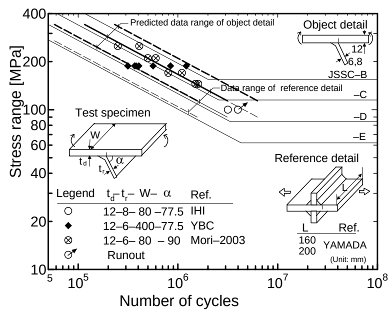

Fig 3: Predicted S-N diagram and fatigue test data

Results from FEA on 2D sub-model in plain strain condition with boundary condition similar to Fig.2 and acting stress quantitatively obtained from panel analysis with shell element indicate that one-mm stresses causing crack under discussion are too small when any types of stresses act at the end of rib side.

An example of prediction of fatigue strength of the detail is shown Fig.3. A few series of fatigue test results obtained from various sources are also plotted for comparison. The graph indicates the good agreement between predicted curves and test results.

Key words: Orthotropic steel deck, trough to deck detail, one-millimeter stress method, FEA, fatigue durability evaluation

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3. Stress range evaluation using an equivalent wheel load

The stress ranges are computed through FEA on two-panel deck model. The model consists of four trough ribs of size U340x250x6 (large width, height, and web thickness, all in mm) placed on the underside of the 12mm deck plate, spanning over three crossbeams spacing 2240mm from each other. Fig.4 shows a shell FE model with meshes 10x10mm near the welded joint of interest. Several models are also generated by increasing thickness of deck plate and trough ribs for comparison.

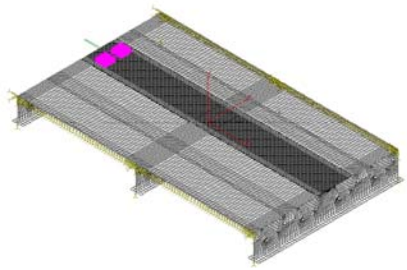

Fig 4: Two-panel deck model

A double tire of 55kN, representing an equivalent wheel load from an actual measurement in Nagoya area, see Fig.6, is used for stress analysis. It acts on two contact areas of 200x200mm and 100mm apart from each other in side direction. The contact area enlargement in 45 degrees in lateral directions is used for simulating effect of stiffness of pavement.

The wheel is assumed to run on a line coinciding with rib wall. This driving line is the one causing highest stress ranges at the joint. It should be pointed out that the bottom face nodal stress in transverse direction at node 10mm apart from intersection of plates is defined as the nominal stress causing crack in deck plate.

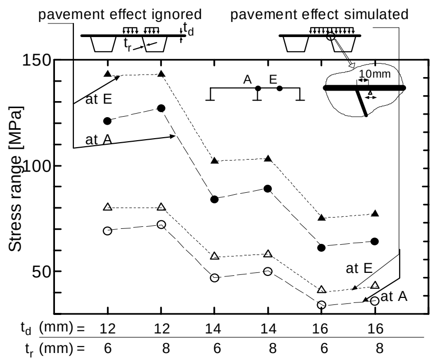

Fig 5: Stress ranges from various two-panel deck models

Some of results are shown in Fig.5. Stress ranges drop around 50% when a deck plate change from 12mm to 16mm; that is, there is a reduction of stress range around 25% as deck plate increases by 2mm (i.e., from 12 to 14mm). There is very small difference when the rib wall changes 2mm (from 6 to 8mm), but it becomes noticeable as deck plate becomes thicker. The stress ranges occurring at span center are higher than at crossbeam around 10 to 20%. Another important result is the stress range reduces by approximately 40% when the effect of pavement stiffness is simulated with increased contact area.

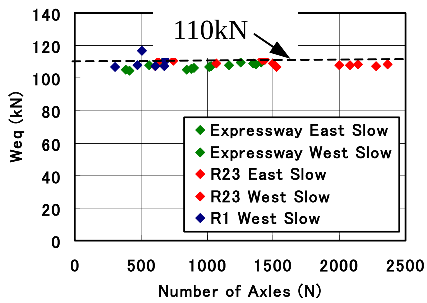

Fig 6: Equivalent axle load from actual measurement in Nagoya

4. Fatigue durability evaluation

Suppose that the basic model is subjected to 1000 equivalent wheel loads per day, and falls in situation discussed in previous section, the joint would fail in 3 years at crossbeam, predicted with the lowest S-N curve and the effect of pavement stiffness is ignored. This figure will become 19 years, if effect of pavement stiffness is taken into account. The same conditions give the quantitative fatigue life of 2 and 12 years at span center. A longer fatigue life is obtained when the deck plate become thicker; for example, 16mm deck and 6mm rib gives a fatigue life of 68 years at span center in case of pavement effect is simulated.

5. Conclusion remark

This approach is rather simple and mostly comparable to conventional procedure, which requires fatigue strength of the detail, stress ranges and its number of cycles. Using shell element to compute stress ranges makes work easier, faster. However, this approach may need future verifications.

Reference:

[1]-Xiao ZG, Yamada K. A method of determining geometric stress for fatigue strength evaluation of steel welded joints. Int J Fatigue 2004; 26(12): 1277-93