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Fatigue tests of welded connections between longitudinal stringer and deck plate in railway bridge orthotropic steel decks

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ABSTRACT

This research presented the fatigue tests of longitudinal stringer-to-deck (SD) welded connections, which have been identified as the locations most sensitive to fatigue damage in the orthotropic steel decks (OSDs) of railway bridges. Four full-scale SD connections were fabricated, and two loading patches were considered. Static loading was first carried out to obtain the structural hot spot stresses at weld toes as well as stress concentration factors (SCFs), by which the hot spots providing the highest stresses were identified. Cyclic loading was then implemented next to the static loading, and the behaviors including fatigue crack initiation and propagation process, fatigue failure mode, characteristic fatigue life, as well as degradation of vertical rigidity, were all obtained from the test. The crack growing process can be totally divided into four stages, and the fatigue lives after the crack arrived at the deck edge were very short. Variations of crack dimensions were also obtained, and the simplified formulae of crack growth rate were numerically fitted so that the crack propagation lives can be predicted by using the crack dimensions. Comparisons also show that the FAT 100 curve in IIW fatigue design recommendation could overestimate the fatigue resistance of such connections where double-sided fillet welds were used to connect the stringer web and the deck plate, and therefore double-sided groove welds with partial or full penetrations are recommended for the stringer-to-deck connections in railway bridge decks.

1. Introduction

Orthotropic steel decks (OSDs), which are composed of deck plates orthotopically strengthened by longitudinal and transverse stiffeners, have been widely used in steel bridges due to their inherent merits. While fatigue cracking is still a prominent problem for such bridge decks since they are directly loaded by the repeated vehicle wheels. Plenty of welds between steel plates, which make the local stresses near the weld toes and roots concentrate to very high levels, are also considered as the critical cause of fatigue resistance deteriorations. Numbers of fatigue cracking damages in OSD bridges have been reported worldwide, including Hu Meng Bridge, Guangdong Province (1997) and Jiang Yin Bridge, Jiangsu Province (1997) in China. Besides highway bridges, OSDs can also been used in railway bridges, and in recent years several high-speed railway bridges incorporated with OSDs, including Nanjing Dashengguan Yangtze River Bridge (2011) and Jinan Yellow River Bridge (2011) in Beijing-Shanghai high-speed railway, Chongqing Hanjiatuo Yangtze River Bridge (2011) in Chongqing-Lichuan railway, and Zhengzhou Yellow River Bridge (2012), have been newly built in China.

The fatigue behavior of railway bridge orthotropic steel decks are obviously different from those of highway bridges due to the differences in structural detailing and cyclic loading. Some achievements regarding fatigue behavior of railway bridge OSDs have been obtained in the past in order to meet the requirements of engineering applications. Frýba et al. [1] investigated the static and dynamic behavior as well as fatigue behavior of railway orthotropic steel bridge deck, and the results showed that well-constructed and welded orthotropic decks are suitable for steel railway bridges. It was also revealed in their further research that the shape of cutouts in the web of the cross-girder, the effect of shear forces and the weld penetrations are the important factors affecting the fatigue properties of orthotropic decks on railway bridges [2]. Li et al. [3] developed a fatigue damage model to simulate the damage accumulation process in bridges subjected to in-field traffic loading and carried out on-line strain gauge measurements on the orthotropic steel deck structure of Tsing Ma Bridge, whose results showed that the fatigue life results derived from the proposed damage model are in good agreement with those obtained from the tests. De Corte et al. [4] assessed the role of ballast in stress reduction on steel orthotropic decks of two railway bridges in Belgium by using the

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continuous high-frequency strain gauge measurements and the finite element method. Pipinato et al. [5] presented a stepwise and practical approach for evaluating the structural integrity of Paderno railway bridge in Italy and performed the fatigue reliability analysis to predict the remaining fatigue life of the bridge deck. Tian et al. [6] established finite-element model of orthotropic steel deck in high-speed railway bridge to simulate the local stresses of typical fatigue details, and results showed that the cut-out in floor beam was the most prone to fatigue damage. Gao et al. [7] compared the stress influence lines of steel box-girder orthotropic deck in highway and railway bridges by using the finite-element method and found that the local stresses at rib-to-floor beam connections in two types of bridges are comparable, despite that the pavements in railway bridge are more helpful in dispersing the stresses in fatigue-sensitive regions than those in highway bridge. Based on stress analysis, Gu et al. [8] established the relationship between the growth speed of fatigue crack and the surrounding stress field in terms of linear elastic fracture mechanics for orthotropic steel decks in railway bridges, and then accomplished the fatigue life predictions of typical cracks on the basis of numerical integration of the Paris law. Chen et al. [9] presented a practical fatigue design approach for highway-railway dual purpose cable-stayed bridges with orthotropic steel deck by considering the fatigue load definition, fatigue detail categorization, global and local structural analysis as well as fatigue strength determination. Song et al. [10] proposed a new theoretical approach for fatigue assessment of a typical welded joint in the orthotropic steel deck of high-speed railway bridges by integrating multiple influencing factors, where the determinative and reliable fatigue-life evaluation of a real bridge was also provided. Wu et al. [11] carried out the finite element analyses of orthotropic steel deck for a self-anchored suspension railway bridge in China, and the fatigue assessments using rain-flow method and nominal stress method were subsequently performed, in which the comparison of effects of cutout form on the stress range were also made. Lu [12] numerically calculated the train wheel induced stress history of typical details in Nanjing Dashengguan Yangtze River railway bridge, and then evaluated the fatigue performance by using Miner's law and S-N curves. More researchers presented the fatigue investigations of welded details in railway bridge steel decks in their dissertations. Li [13] and Wang [14] conducted the fatigue tests of rib-to-diaphragm connections in railway orthotropic steel bridge deck, where the influence factors have been identified. Yu [15] numerically analyzed the stresses distributions within the local areas of welded connections in high-speed railway bridge OSDs and then experimentally investigated the fatigue resistances of rib-to-deck connections at diaphragm. Compared with abundant achievements concerning highway bridge OSDs that have been presented in the documents such as IIW fatigue design recommendations [16], AASHTO LRFD bridge design specifications [17], and FHWA manual [18], existing fatigue studies with regard to railway bridge OSDs are far from complete. The fatigue research, especially the experimental fatigue investigation, of typical welded connections in railway bridge OSDs are rarely reported in published literatures, which limits the applications of OSD in railway bridges.

In railway bridge OSDs, besides the longitudinal U-ribs and transverse diaphragms that have been commonly used in highway bridge decks, the longitudinal stringers, which locate below the deck plate and align with the rails, are usually employed as the most important stiffeners of the deck plate so that the bearing resistances to train wheel loads can be significantly enhanced. Fig. 1 shows the typical details of railway OSD in Nanjing Dashengguan Yangtze River Bridge. These stringer-to-deck welded connections have been identified as the most fatigue sensitive locations since the hot spot stresses around these areas are of the highest within the whole deck. The reason lies in that the connections are directly under the repeat loading of train wheels. The fatigue cracking or fracture of such connections could lead to destructive deteriorations in the performances of the deck. Therefore, this research carried out high-cycle fatigue tests to investigate the fatigue failure process and cracking characteristics of stringer-to-deck welded connections in orthotropic steel decks of railway bridges. Structural hot spot stresses and fatigue behaviors including the failure pattern, characteristic fatigue life, crack initiation and propagation process, as well as the variations of crack dimension and joint rigidity were obtained and discussed.

2. Test setup

2.1. Specimens

Four full-scale stringer-to-deck (SD) welded connections were fabricated and tested. The structural dimensions of four specimens (i.e., SD-1, SD-2, SD-4, SD-4) were all identical to the details in Nanjing Dashengguan Yangtze River Bridge. In order to simulate the real structures, two U-shape ribs adjacent to the stringer were also introduced into each specimen. The thickness of deck plate t_d is 16 mm. The T-shape stringer has a web plate of 12 mm thick and 472 mm high, as well as a bottom flange of 12 mm thick and 240 mm wide. The thickness of U-rib plate is 8 mm. All ribs have the height of 260 mm and the top-width of 300 mm. The specimens are totally 1900 mm wide in the transverse direction and 400 mm long in the longitudinal direction. The stringer's web plate was welded to the deck plate through double-side fillet welds without penetration, which is also identical to the welds in practical structure. All welds were produced by carbon dioxide gas arc welding, which is in strict accordance with Chinese welding code [19]. The T-shape stringer was first welded to the deck, followed by the implement of welds between U-rib and deck. Before testing, all specimens have been beaten and vibrated by man hand so that the internal welding residual stresses could be reduced. Fig. 2 shows the geometries of the connection.

The steel plates used for manufacture were of Q345qD grade ("345" represents the designated yield strength, "q" represents the application to bridges, "D" represents the quality grade corresponding to the required impact toughness at −20 °C), which strictly conforms to GB/T 714-2008 [20]. Standard coupons were taken from the used plates and then tested in uniaxial tension according to GB/T 228-2002 [21]. The measured mechanical properties of steels are provided in Table 1.

The authors conducted the FE analysis of the whole deck and found that the longitudinal influence lines for the local stresses at SD connections are shorter than the minimum distance between adjacent axles (i.e., 2.5 m) of a standard CRH train which pass through the bridge most frequently. Fig. 3 demonstrates the load scheme of a CRH train. Therefore, in order to investigate the fatigue failure process of the connections, one of eight wheel loads of CRH380A train was introduced, while the load ranges were different for each specimen so that diverse hot spot stress ranges can be considered. Two loading patches, i.e., 250 mm (longitudinal) by 250 mm (transverse) for specimens SD-1 and SD-2, and 250 mm (longitudinal) by 510 mm (transverse) for specimens SD-3 and SD-4, were considered in order to investigate the possible influence of transverse loading on fatigue resistances of both stringer-to-deck welds and rib-to-deck welds. The specimens were simply supported at the two ends of deck plates. The boundary conditions have been validated by comparing the stresses of specimen with the stresses of whole deck structure by the use of finite element method. Results show that the stress distributions of two models are very similar, although the stress values could be somewhat different. The applied load patches and the employed boundary conditions are shown in Fig. 2.

2.2. Test rig

Before the test, two steel plates of 30 mm thick, upon which two I-section steel columns have been vertically erected, were fixed to the foundation by using the fastening bolts. The specimens were then simply supported on the top surfaces of I-section steel columns via the