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Q1 Fatigue evaluation of a composite railway bridge based on fracture mechanics through global–local dynamic analysis

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ABSTRACT

An enhanced fatigue assessment of critical welded details in a steel–concrete composite railway bridge was carried out by fatigue crack propagation analysis based on linear elastic fracture mechanics (LEFM). The most fatigue critical connections concerned in this study were identified by the preliminary fatigue assessment based on the *S-N* method in the previous research of the authors Zhou et al. (2013). Three-dimensional crack models of the critical connections were incorporated into the global–local finite element model of the bridge. The stress intensity factor (SIF) histories of the cracks were calculated through dynamic analysis of the bridge due to high-speed train passages, which validated the applicability and the accuracy of the empirical SIF formulas for the concerned bridge details. The fatigue crack growth curve, represented by crack size versus number of train passages, was obtained through crack propagation analysis based on the Paris law and LEFM, and fatigue propagation life was predicted for each critical connection. The proposed crack propagation analysis method provides a general and alternative approach to evaluate the fatigue life of welded details in steel bridges.

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1. Introduction

In the current design codes for steel structures, such as Eurocode 3 [2], BS 5400 [3] and AASHTO [4] etc., the prevalent fatigue design and assessment method is based on the detail category specified *S-N* curve and the Palmgren–Miner [5–6] cumulative damage rule. If the necessary test data in the form of fatigue strength curves are available for the specific detail categories, it is simple and efficient to use the nominal stress based *S-N* method to conduct fatigue design and assessment of a steel bridge. However, in reality, *S-N* curves are only available for a limited number of structural details, which are given by the classification tables in the design codes [2–4]. Complex structural details that are excluded from the code specified fatigue categories, or multi-axial stress conditions that cause difficulties in determination of the nominal stresses, invalidate the applicability of the nominal stress based *S-N* method. Furthermore, weld defects, such as welding cracks, incomplete penetrations, inclusions, undercuts, etc., are unavoidable in welded details of steel bridges. The traditional *S-N* method is inapplicable to evaluate the effect of a specific defect on the fatigue life, while linear elastic fracture mechanics (LEFM) [7–8] can be used to deal with fatigue crack propagation analysis of cracked welded details.

When a fatigue crack is detected in a steel bridge in service, the remaining fatigue life of the cracked detail needs to be estimated. The LEFM based approaches are able to provide knowledge about crack size and crack growth rate under actual service loads. With this information, it can be decided that either the crack propagates slowly so that enough time is available before retrofitting or the crack propagation is accelerated and the time for repair, retrofitting or replacement is short. Furthermore, the crack propagation information predicted by the LEFM approaches also helps to set up a plan for regular inspection and suitable repair measures before the crack grows to a critical size. The application of the LEFM in fatigue crack propagation analysis becomes more and more widely accepted [9–14]. Most of the researches on the LEFM based fatigue crack propagation analyses were concentrated on the local structural details and the solutions for the stress intensity factor (SIF) around the crack front [9–10], while only a few efforts [13–14] were made to determine the surrounding stress or deformation histories of the local details which drive the crack propagation, and even few studies deal with the localized crack growth model within a global bridge structure. In reality, fatigue crack propagation will change the stiffness and the stress state of the local structural detail which will further affect the crack growth behavior in turn. This interaction problem requires a global–local cracked model of the structure to implement crack propagation analysis, or an isolated local cracked model whose boundary conditions are not affected by local changes in stiffness due to crack propagation.

In the present study, an enhanced fatigue assessment of the critical welded connections in a steel–concrete composite railway bridge, the

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Sesia viaduct, was performed by the LEFM based crack propagation analysis through dynamic simulations of the global–local bridge model. The main purpose of this study is to predict the fatigue crack propagation life of the most fatigue-vulnerable welded connections in the bridge, as well as to compare the fatigue life estimated by the LEFM approach and the nominal stress based *S-N* method in the previous research [1]. In the previous study of the authors [1], a global finite element (FE) model of the Sesia viaduct was built and validated by the in-situ dynamic test results including modal identifications and strain measurements [15]. The dynamic stress histories of the concerned structural details generated by the global FE model of the bridge due to a high-speed train passage were used to calculate the fatigue damage based on the *S-N* method and then, the most fatigue critical welded connections were identified. As a continuation, in this study, the most fatigue critical connections were built with brick elements, which were incorporated into the global FE model. This FE model is termed as the global–local model of the bridge in the following paragraphs. The SIF histories of the cracks in the critical connections were obtained by global–local dynamic analyses of the bridge, which were used to validate the empirical SIF formulas for the pertinent connections. Then, the fatigue propagation life of each critical connection was predicted using a standard Paris crack growth model based on the validated SIF equations.

2. Global bridge model and fatigue critical connections

2.1. Sesia viaduct and dynamic experiments

The Sesia viaduct was constructed along a new Italian high-speed railway line connecting Torino with Milano, over the Sesia River near the city of Novara. It is a box-girder steel–concrete composite bridge and consists of seven simply supported spans, each approximately 46 m long, reaching a total length of 322 m. A general view of the bridge is given in Fig. 1. The double-box steel girder has 13 intermediate and 2 end cross frame diaphragms (see Fig. 2). The bottom steel box made of S355 steel was built by in-situ assembly of three segments, each roughly 15 m in length, connected by full penetration butt welding. The thickness of the lower flanges in segment C1 is 25 mm, which is smaller than that in segment C2, 30 mm. However, the thickness of the girder webs A, B and C in segment C1, 20 mm, is larger than that in segment C2, 18 mm. The top concrete slab cast in the field on prefabricated concrete elements is 13.6 m wide and 0.4 m thick, and is connected to the upper flanges of the steel box girder by shear studs. On the ballast, two tracks with UIC60 rails are supported by prestressed concrete sleepers at an interval of 0.6 m.

In order to evaluate the dynamic behavior, the actual train loads and the fatigue stress spectra of the bridge, preliminary dynamic experiments and long-term monitoring were carried out on the Sesia viaduct [15–17]. Due to the large scale of the bridge, in-situ measurements were concentrated on the second span from the Torino side. Besides, a small number of accelerometers were placed on the first and third spans to evaluate the dynamic interaction between two adjacent spans. The vertical and/or horizontal accelerations at 89 positions were measured with 33 accelerometers in several different setups, where 7 fixed reference positions including 1 horizontal and 6 vertical acceleration

measurements were adopted. A full description and arrangement of accelerometers can be found in Ref. [15]. Measurements were performed to obtain acceleration responses of the bridge due to ambient vibrations (excited by wind and traffic load of the adjacent highway) and high-speed train passages. With the vibration data, modal identification of the bridge (eigenfrequencies, damping ratios and mode shapes) was performed by the reference based stochastic subspace identification [18]. Based on the ambient vibration data, a total of 8 modes were identified for the second span, including the first and second bending modes, as well as the first and second torsional modes. According to the mode shapes of the adjacent spans, the identified modes of the second span could be subdivided into symmetrical and anti-symmetrical ones, which revealed that the neighboring spans were dynamically coupled through the ballast layer and the continuous rails. From the free vibration data immediately after the train left the second span, three symmetrical modes, i.e. the first and second bending modes and the first torsional mode, were extracted. Additionally, the first and second symmetrical bending modes were also identified through the vibration data during a train crossing the span. Thus, it was concluded that the train passage predominately excited the symmetrical modes. Detailed modal analysis results were described and discussed in Refs. [15,19].

The dynamic strains in the longitudinal direction were measured with the fiber optic strain sensors during a high-speed train passage. These sensors measured the relative displacement between a pair of points at a distance of 1.0 m on the steel box girder or 1.2 m on the concrete sleepers. A total of 16 sensors were installed at Cross Sections 1 and 5. A schematic plot of the strain sensor arrangement can be found in Ref. [1].

2.2. Global model of a single span and verification

Due to the large dimensions of the Sesia viaduct including seven spans, a FE model of the complete bridge would lead to expensive computational costs. As the seven simply supported spans are identical to each other, a single span model is sufficient and also efficient to estimate the fatigue damage of the bridge. Therefore, a detailed FE model of the second span (as shown in Fig. 3), was built using the general FE software ABAQUS [20], and appropriate boundary conditions were applied at the ends of rails and ballast to simulate the weak coupling between the adjacent spans. The steel box girder was modeled by four-node shell elements S4 and the bracings were simulated by beam elements B31 in combination with shell elements S4 at the end connections. Couplings of 6 degree of freedoms (DOFs) were adopted to join the endpoint of beam element with the edge of shell element, at a distance of 0.8 m from the bolt connections of the bracings. The preloaded high-strength bolt connections of the bracings and the gusset plates were assumed rigid and realized by the TIE option in ABAQUS [20], which is a surface-based constraint to make all DOFs equal for a pair of surfaces. The concrete deck and safety barrier were modeled by eight-node hexahedral brick elements C3D8. The shear studs were not explicitly modeled and the TIE constraints were used to connect the concrete deck to the upper flanges of the steel box girder. The ballast layer was assumed as a continuum and simulated by brick elements C3D8. The

(a)

(b)

Fig. 1. Sesia viaduct: (a) general view of the spans and (b) interior view of the double-box girder.

Fig. 2. Layout of a simply supported span of the Sesia viaduct: (a) elevation view and (b) plan view.

global FE model included about 1800 beam elements, 84,000 shell elements and 6400 brick elements, resulting in over 85,500 nodes.

Each railway track model was composed of two beams representing the rails and a series of mass points representing the concrete sleepers. The rail pads that fasten the rails to the sleepers were simulated by springs & dashpots between the beams and the mass points, with stiffness $K_{rp} = 500$ MN/m and damping coefficient $C_p = 200$ kNs/m [15]. The vertical bending stiffness of the rail is 6.4×10^6 N·m² and the linear density is 60 kg/m. Symmetrical boundary conditions were used at the ends of rails and ballast layer, as symmetrical modes were predominantly excited by the train passage in the experiments. The bearing

system was composed of one unidirectional sliding bearing, two fixed bearings and three bidirectional sliding bearings. At the lower flanges of the steel box girder in the FE model, the corresponding translational DOFs were restrained in accordance with the bearing system as shown in Fig. 3.

During the preliminary dynamic tests, there was only one type of high-speed train ETR500Y running on the newly-built railway. Therefore, this particular train ETR500Y was adopted in the FE dynamic simulations of the bridge due to train passages. The axle load scheme of the train ETR500Y could be found in Ref. [1], which was provided by the Italian railway company RFI S.p.A. In the FE analyses, a moving load model,

Fig. 3. Global FE model and bearing system of the second span.

in which the train axles were represented by a series of moving constant forces, was adopted to simulate the train passage through the span. An ABAQUS user subroutine DLOAD [20] coded by FORTRAN was used to realize the moving load model of the train.

The natural frequencies of the bridge span were obtained through modal analysis of the FE model as shown in Fig. 3. In modal identification based on the in-situ measurements, the frequencies were identified from the ambient vibration, the train induced vibration and the free vibration after the train passage, respectively. The frequencies extracted from the ambient vibration data were adopted to validate the accuracy of the FE model in predicting the fundamental dynamic characteristics of the bridge span. The frequencies obtained by FE analysis and identified from the ambient vibration data were in good agreement which was presented in the previous work of the authors [1].

In the dynamic analysis, a structural damping matrix was assumed according to the Rayleigh damping formulation [21], which is proportional to the mass matrix and the stiffness matrix. The mass and stiffness damping coefficients α and β can be estimated by two reference frequencies and the associated modal damping ratios. Here, the natural angular frequencies of the first bending and the first torsional modes of the span were adopted as the reference frequencies. An identical modal damping ratio 2.5% was considered for both reference modes, which was nearly the mean value of all identified modal damping ratios in the tests [1].

In the dynamic experiments, the train ETR500Y was running at a speed of 288 km/h from Milano to Torino on the AB-side of the double track railway as indicated by Fig. 3. The dynamic responses of the bridge span during and after the train passage with the actual speed were computed by implicit time integration method in ABAQUS/Standard. Referring to the previous work [1], the correspondence between the measured and simulated acceleration results both in time and frequency domains was reasonably good, which indicated that the dynamic behavior of the bridge span was well represented by the global FE model. The computed time histories of the longitudinal displacements of the nodes corresponding to the positions of the fiber optic sensors were extracted for the strain history calculations. The good agreement in simulated and measured strain history results [1] further validated that the dynamic response due to a high-speed train passage was accurately predicted by the aforementioned global FE model, which could be adopted in subsequent fatigue assessment of the bridge.

2.3. Fatigue critical connections predicted by S-N method

The preliminary fatigue assessment of the Sesia viaduct was performed in the previous study of the authors [1] using the traditional S-N method, which is based on the structural detail category determined S-N design curve and the nominal stress history applied on the concerned structural detail. Firstly, several structural details of the Sesia viaduct were selected for fatigue evaluation, including welded and bolted connections. The selected structural details were classified into the most relevant fatigue detail categories according to the current design codes, i.e. Eurocode 3 [2], BS 5400 [3] and AASHTO [4]. Each category was specified with a standard S-N curve in each design code. Secondly, dynamic responses of the bridge span due to one passage of the train ETR500Y were simulated by the validated FE model and the nominal stress histories at the concerned structural details were monitored and extracted from the FE analysis results. Thirdly, the variable amplitude stress histories of the structural details were converted into stress range spectra (stress range versus number of cycles) using the Rainflow cycle counting method [22] and then the fatigue damage of each structural detail was calculated based on the Palmgren–Miner [5–6] cumulative damage rule and the design code specified S-N curve. Based on these analyses, a fatigue criticality ranking of each structural detail was obtained and the most fatigue critical connections were identified from all concerned connections of the steel box girder.

Through the preliminary fatigue assessment based on the traditional S-N method, the most fatigue critical connection within each structural detail category was identified in the previous work [1]. Among the cope hole detail as shown in Fig. 4a, CH-A05&06 on Girder A between Cross Sections 5 and 6 is the most critical connection of all connections of this category in the bridge span. For the assembling butt welds of the three segments of the bridge span, butt weld connection BW-05&06 (Fig. 4b) on the lower flanges of the steel box girder between Cross Sections 5 and 6 is the most fatigue critical. Among the non-load carrying fillet welds of the transverse stiffeners attached to the lower flanges of the steel girders, ST-A05 (Fig. 4c) of Girder A at Cross Section 5 is the most critical connection all through the span. Of all load carrying fillet welds around the gusset plates of the bracings, connection FW-15W8 (Fig. 4d) at Cross Section 15 on the transverse stiffener of Girder C is predicted to be the most fatigue critical. In these welded connections, weld defects, such as welding cracks, lack of fusion, incomplete penetration, inclusions, undercuts and so on, may be unavoidable. In case of such conditions, the traditional S-N method may not be able to quantitatively evaluate the effect of weld defects on the fatigue life. Therefore, a further fatigue assessment of these critical welded connections is performed by fatigue crack propagation analysis based on the LEFM approach in the following sections.

3. Cracked welded details and stress intensity factor

3.1. Empirical solution for stress intensity factor

Fatigue failure is a process that involves two stages of damage, i.e. crack initiation and crack propagation. In welded connections, the fatigue crack initiation life is generally quite short and could be neglected, as welded details probably include small crack-like defects, such as lack of fusion, inclusions, undercuts and so on. Therefore, the total fatigue life of a welded detail could be assumed as a function of the crack propagation life [23]. In crack propagation analysis based on the empirical Paris law [24], the controlling parameter, i.e. the stress intensity factor (SIF) range ΔK , needs to be determined. According to the locations, shapes and sizes of the crack-like flaws that might exist in the welded details of the Sesia viaduct, simplified crack models with through edge crack, semi-elliptical surface crack and central through crack were assumed for the critical welded connections and the empirical formulas for SIF calculations were obtained by modifications of the theoretical crack models.

For mode I crack, corresponding to in-plane tensile crack opening, the SIF for arbitrary loading may be expressed in the following general form [8]:

$$K_I = Y\sigma\sqrt{\pi a} \quad (1)$$

where, Y is a geometry correction factor and depends on the crack configuration and the type of loading; σ is the remotely applied stress; and a is the crack size.

For a semi-elliptical surface crack in a finite width plate subjected to tension as shown in Fig. 5, an empirical solution for SIF was given by Newman and Raju [25–26] as follows:

$$K_I = Y\sigma\sqrt{\pi a} = M_k f_w \sigma \sqrt{\pi a} \quad (2)$$

where, the geometry correction factor Y is decomposed into M_k and f_w ; M_k is the stress magnification factor given by Eq. (3); f_w is the finite width correction factor given by Eq. (10); and a is the crack depth as shown in Fig. 5.

$$M_k = \left[M_1 + M_2 \left(\frac{a}{t} \right)^2 + M_3 \left(\frac{a}{t} \right)^4 \right] g f_\phi / Q \quad (3)$$

Fig. 4. Close-up views and details of the fatigue critical connections identified by the traditional S-N method: (a) CH-A05&06; (b) BW-05&06; (c) ST-A05 and (d) FW-15W8.

where,

$$M_1 = 1.13 - 0.09 \left(\frac{a}{c} \right) \quad (4)$$

$$M_2 = -0.54 + \frac{0.89}{0.2 + (a/c)} \quad (5)$$

$$M_3 = 0.5 - \frac{1}{0.65 + (a/c)} + 14 \left(1 - \frac{a}{c} \right)^{24} \quad (6)$$

$$g = 1 + \left[0.1 + 0.35 \left(\frac{a}{t} \right)^2 \right] (1 - \sin \phi)^2 \quad (7)$$

$$f_\phi = \left[\left(\frac{a}{c} \right)^2 \cos^2 \phi + \sin^2 \phi \right]^{1/4} \quad (8)$$

$$Q = \left[1 + 1.464 \left(\frac{a}{c} \right)^{1.65} \right]^{1/2} \quad (9)$$

$$f_w = \left[\sec \left(\frac{\pi c}{2W} \sqrt{\frac{a}{t}} \right) \right]^{1/2} \quad (10)$$

In Eqs. (2) to (10), the definitions of the geometries W , t , a , c and ϕ are shown in Fig. 5 and the formulas are effective only when the following conditions are satisfied, i.e. $0 < a/c \leq 1.0$, $0 \leq a/t < 1.0$, $c/W < 0.5$ and $0 \leq \phi \leq \pi$.

The steel box girder of each bridge span was constructed by in-situ assembly of three segments connecting by complete joint penetration

Fig. 5. Stress intensity factor SIF calculation for a semi-elliptical surface crack in a finite plate subjected to combined tension and bending.

butt welding. For the fatigue critical connection BW-05&06 at the lower flange of the steel box girder between Cross Sections 5 and 6 (referring back to Fig. 4b), a semi-elliptical surface crack model (shown in Fig. 6) is assumed for the butt weld connection. In consideration of the local stress concentration raised by the geometric irregularity of the butt weld, the SIF magnification factor M_{km} is introduced by the British Standard BS 7910 [27] to modify the Newman-Raju solution given in Eq. (2) by multiplication. For point A at the crack front shown in Fig. 6, the expression of the SIF magnification factor M_{km} for tension is given by Eq. (11) [27]:

$$M_{km} = u_1 \left(\frac{a}{t} \right)^{k_1} \quad (11)$$

where, a is the crack depth; t is the plate thickness; parameters u_1 and k_1 are given in BS 7910 [27], which are related to the crack depth a and the weld geometries, i.e. the butt weld width L and the plate thickness t .

In the Sesia viaduct, transverse stiffeners were attached to the steel box girders by fillet welding. For the fatigue critical connection ST-A05 on Girder A at Cross Section 5 (referring back to Fig. 4c), a semi-elliptical surface crack is assumed in the lower flange near the fillet weld toe of the transverse stiffener as shown in Fig. 7. The SIF magnification factor M_{km} is included to modify the Newman-Raju solution by multiplication, so as to take the geometric effect of the fillet weld into account. For point A at the deepest position of the surface crack, the expression of the SIF magnification factor M_{km} is given as follows [28]:

$$M_{km} = u_2 \left(\frac{a}{t} \right)^{k_2} \quad (12)$$

where,

$$u_2 = 0.8068 - 0.1554 \left(\frac{V}{t} \right) + 0.0429 \left(\frac{V}{t} \right)^2 + 0.0794 \left(\frac{W}{t} \right) \quad (13)$$

Fig. 6. A semi-elliptical toe crack model for the butt weld connection at the lower flange of the steel box girder.

Fig. 7. A semi-elliptical surface crack model for the fillet weld connection of the transverse stiffener.

$$k_2 = -0.1993 - 0.1839\left(\frac{v}{t}\right) + 0.0495\left(\frac{v}{t}\right)^2 + 0.0815\left(\frac{w}{t}\right) \quad (14)$$

where, v and w are the vertical height and the width of the fillet weld as shown in Fig. 7; and t is the plate thickness of the lower flange of the steel box girder.

In assembly of the three segments of the steel box girder, cope holes were drilled in the web-flange transition regions to allow continuous butt welding of the flanges and to eliminate the triple point welds. For the fatigue critical connection CH-A05&06 on Girder A at Cross Sections 5 and 6 (referring back to Fig. 4a), a through edge crack is assumed in the butt weld connection at the apex of the cope hole as shown in Fig. 8. In this case, the general SIF equation given in Eq. (1) can be further rewritten as follows [9,29]:

$$K_I = Y\sigma\sqrt{\pi a} = M_k M_{km} \sigma \sqrt{\pi a} \quad (15)$$

where,

$$M_k = 1.1216 + 6.52\left(\frac{a}{W}\right)^2 - 12.39\left(\frac{a}{W}\right)^4 + 89.06\left(\frac{a}{W}\right)^6 - 188.61\left(\frac{a}{W}\right)^8 \quad (16)$$

$$M_{km} = 1.3793\left(\frac{a}{R}\right)^{-0.2627} \quad (17)$$

where, R is the radius of the cope hole.

In the load carrying fillet welds around the gusset plates of the diagonal bracings, FW-15W8 at the bridge bearing near Girder C (referring back to Fig. 4d) is the most fatigue critical connection and a root crack

representing the incomplete penetration is modeled for the fillet weld connection, as shown in Fig. 9. For tensile loading perpendicular to the fillet weld, the SIF solution for the root crack tip A is given as follows [27]:

$$K_I = Y\sigma\sqrt{\pi a} = M_k f_{wm} \sigma \sqrt{\pi a}. \quad (18)$$

The influence of joint geometry on the SIF for a root flaw in a fillet weld is accounted for by the application of the modified finite width correction factor f_{wm} given in Eq. (19) and the stress magnification factor M_k given by Eq. (20) [27,30].

$$f_{wm} = \sqrt{\sec\left[\frac{\pi}{2}\left(\frac{2a}{W}\right)\right]} \quad (19)$$

$$M_k = \lambda_0 + \lambda_1\left(\frac{2a}{W}\right) + \lambda_2\left(\frac{2a}{W}\right)^2 \quad (20)$$

where,

$$\lambda_0 = 0.956 - 0.343\left(\frac{h_f}{B}\right) \quad (21)$$

$$\lambda_1 = -1.219 + 6.210\left(\frac{h_f}{B}\right) - 12.220\left(\frac{h_f}{B}\right)^2 + 9.704\left(\frac{h_f}{B}\right)^3 - 2.741\left(\frac{h_f}{B}\right)^4 \quad (22)$$

$$\lambda_2 = 1.954 - 7.938\left(\frac{h_f}{B}\right) + 13.299\left(\frac{h_f}{B}\right)^2 - 9.541\left(\frac{h_f}{B}\right)^3 + 2.513\left(\frac{h_f}{B}\right)^4. \quad (23)$$

In Eqs. (31) to (23), the definitions of the geometries are shown in Fig. 9 and the ranges of application are as follows, $0.1 \leq 2a/W \leq 0.7$ and $0.2 \leq h_f/B \leq 1.2$.

3.2. Stress intensity factor calculated by global–local FE model

As the configurations of the fatigue critical connections shown in Fig. 4 are somewhat complicated, the stress fields around the connections are not uniformly distributed. This makes difficulty in the determination of the nominal stresses. Furthermore, the exact location of the crack plays an important role in calculation of the SIF at the crack front. Therefore, a refined local model for each fatigue critical connection containing an initial crack was incorporated into the global FE model described in Section 2.2. The SIF histories of the crack front were calculated directly through dynamic analysis of the global–local cracked FE model of the bridge span. These results are used to validate

Fig. 8. A through edge crack model for the butt weld connection at the apex of a cope hole.

Fig. 9. A root crack model for fillet weld connection around the gusset plate of the diagonal bracing.

the applicability and the accuracy of the empirical formulas for SIF solutions of cracks in the critical welded details.

As shown in Fig. 10, the fatigue critical connections identified by the S-N method, i.e. the cope hole detail CH-A05&06, the butt weld connection BW-05&06, the fillet weld attachment detail ST-A05 and the load carrying fillet weld connections FW-01W2, FW-01W8, FW-15W2 and FW-15W8, were refined with three-dimensional brick elements and incorporated into the global shell model of the steel box girder. The local cracked model with brick elements was connected to the global shell element model by the SHELL-TO-SOLID COUPLING constraints [20] in ABAQUS. The geometry of the crack was modeled according to the configuration given by Figs. 6 to 9. The detailed FE meshes of the cracked welded connections are shown in Fig. 11.

As shown in Fig. 11a, a through edge crack with a crack depth of a , was introduced into the butt weld at the apex of the cope hole. Through the web thickness, 4 elements were meshed. Around the crack front, 7 contours were considered, adopting 24 elements in each of them. As shown in Fig. 11b, a semi-elliptical surface crack (with the width $2c$ and the depth a) was included at the toe of the assembling butt weld on the lower flange of the steel box girder and the crack center was situated at a distance of 680 mm from the web, where the largest longitudinal stress of the butt weld was sustained. As the connection CH-A05&06 and BW-05&06 were located at the lower flange of the steel box girder near the midspan, the associated through edge crack and semi-elliptical surface crack were primarily in mode I (opening) tensile loading. The refined local crack models, as shown in Fig. 11a and b, contained more than 19,000 brick elements, including hexahedral elements C3D8, wedge elements C3D6 and tetrahedral elements C3D4.

As shown in Fig. 11c, a semi-elliptical surface crack (with the width $2c$ and the depth a) was inserted into the fillet weld toe near the edge of the transverse stiffener where the largest longitudinal stress was predicted by the global shell element model. The semi-elliptical surface crack was mainly in mode I loading, as the connection ST-A05 was also located at the lower flange close to the midspan of the simply supported bridge span. The refined local FE model as shown in Fig. 11c consisted of 11,000 brick elements.

As shown in Fig. 11d, central through cracks (i.e. fillet weld root cracks) were modeled to simulate the incomplete penetration gaps between the gusset plates of the diagonal bracings and the lower flanges of the steel box girder. Four such connections at the bridge bearings (shown in Fig. 10), i.e. connections FW-01W2, FW-01W8, FW-15W2 and FW-15W8, were refined with local crack models and each local

model contained 20,000 brick elements. When the train runs on the FW-15W2 to FW-01W2 side as indicated by Fig. 10, the diagonal bracings of this side are in compression and the fillet weld root cracks of the associated gusset plates are closed, while the diagonal bracings of the other side are in tension and the corresponding cracks are open. Those can be reflected by the SIF results of the crack fronts in the subsequent FE analyses.

The definitions of the above mentioned cracks were carried out in the INTERACTION MODULE (special → crack) [20] in ABAQUS/CAE and the contour integrals around the crack fronts were calculated to evaluate the SIF, which were defined by the history output request in the STEP MODULE [20]. Dynamic responses of the bridge span due to the train ETR500Y passage were re-computed through the global–local FE model and the SIF histories of the crack fronts were obtained for each cracked welded connection. The SIF results indicate that: (a) for connection BW-05&06 and ST-A05, the largest value is at the deepest point of the semi-elliptical surface crack; (b) for connection CH-A05&06, the largest value appears at the center of the through edge crack; and (c) for connection FW-15W8, SIF varies along the crack front as the stress distributes nonuniformly along the fillet weld. The computed time histories of the maximum mode I stress intensity factor K_I around the crack front of each crack with a certain size a and aspect ratio a/c are presented in Fig. 12.

It is well known that the crack size is relatively small in the earlier stage of crack propagation but obviously affects the fatigue crack propagation life. Therefore, relatively small crack sizes, i.e. $a = 0.2, 1.0$ and 5.0 mm, were selected to validate the empirical SIF solutions using the global–local FE analyses. For a semi-elliptical initial flaw at the weld toe, an aspect ratio $a/c = 0.2$ is recommended by the British Standard BS 7608 [31] and thus, was adopted in the present validation analyses. Comparisons between the empirical solutions and the global–local FE analyses for the SIF calculations are given in Table 1, where $K_{I,emp}$ is calculated by the empirical formulas using the maximum nominal stress σ_{max} of the fatigue critical connection, which was generated by the dynamic analysis of the global shell element model, and $K_{I,FEM}$ is the largest SIF value along the crack front from the entire loading history, which was obtained directly by the dynamic analysis of the proposed global–local cracked FE model. As indicated by Table 1, the error between the empirical solution and the global–local FE model calculation in SIF, ranges from -10.5% to $+9.2\%$, which is probably due to the differences between the nominal stress calculated by the global FE model and the real stress field analyzed by the present global–local FE model. In

Fig. 10. Global–local FE model of the second span of the Sesia viaduct.

Fig. 11. Local crack models: (a) CH-A05&06; (b) BW-05&06; (c) ST-A05 and (d) FW-15W8.

general, an acceptable agreement is achieved between the empirical solutions and the global–local FE model analyses, which validates the applicability and the accuracy of the empirical SIF solutions for the cracked welded connections.

4. Fatigue crack propagation analysis

4.1. Material properties, initial and critical crack size

In the steel box girder of the Sesia viaduct, structural steel S355J2 was adopted when the plate thickness was less than 40 mm, while S355K2 was applied when the thickness exceeded 40 mm. The plate thickness in the aforementioned fatigue critical connections ranged from 18 mm to 30 mm, thus, according to the steel products standard EN 10025 [32], the yield strength f_y and the ultimate tensile strength f_u of the structural steel S355J2 were 355 MPa and 510 MPa, respectively, and the minimum Charpy V-notch impact energy A_{kv} at -20°C was 27 joules. Because of missing the test data of the material fracture toughness, an empirical correlation as shown in Eq. (24), recommended by BS 7910 [27], was used to estimate the fracture toughness of the structural steel S355J2 by the minimum Charpy energy:

$$K_{mat} = \left[(12\sqrt{A_{kv}} - 20)(25/t)^{0.25} \right] + 20 \quad (24)$$

where, K_{mat} (in $\text{MPa}\cdot\text{m}^{1/2}$) is the estimated fracture toughness; A_{kv} (in joules) is the lower bound Charpy V-notch impact energy at the service temperature; and t (in mm) is the thickness of the material for which an estimate of K_{mat} is required.

The fatigue life of the studied critical welded connections based on the crack propagation analysis was determined using a crack growth relationship proposed by Paris and Erdogan [24], known as the Paris law. The Paris law parameters $C = 2.1111 \times 10^{-15} \text{ MPa}\cdot\text{mm}^{1/2}$ and $m =$

3.74 in this study were defined after the fatigue crack growth rate tests on steel S355 performed by De Jesus et al. [33], which were the average results of the three test series with different stress ratios $R_\sigma = 0.25, 0.50$ and 0.75 . The threshold stress intensity factor range, $\Delta K_{th} = 63 \text{ MPa}\cdot\text{mm}^{1/2}$, given by BS 7910 [27], was applied in the crack propagation analyses of the critical welded connections in this study.

The initial crack size and shape are important parameters in crack propagation analysis and will highly affect the outcome of the remaining fatigue life estimation. In the existing steel structures, non-destructive inspection, such as ultrasonic test, is generally accepted as a reliable way to determine the size, shape and location of the initial flaw. When flaws or cracks haven't been detected by ultrasonic test, the initial crack size may be assumed conservatively according to the maximum detection ability of the equipment. The non-destructive inspection data of the Sesia viaduct were unavailable and therefore initial cracks were introduced by reasonable assumptions. As given in Table 2, the initial crack size and shape of the studied critical welded connections were assumed according to the similar welded details in literatures. When the actual crack size cannot be determined in the welded details, a reasonable assumed initial size a_0 , ranging from 0.1 to 0.25 mm, is recommended by BS 7608 [31]. A certain crack size $a_0 = 0.15 \text{ mm}$ is recommended by the International Institute of Welding [34]. From the perspective of LEFM, El Haddad et al. [35] proposed a model to express the threshold stress intensity factor range ΔK_{th} using the fatigue limit stress range $\Delta\sigma_D$ and a fictitious initial crack a_0 . Therefore, a fictitious crack length could be estimated as follows:

$$a_0 = \frac{1}{\pi} \left(\frac{\Delta K_{th}}{Y \cdot \Delta\sigma_D} \right)^2 \quad (25)$$

where, Y is an all-including geometry correction factor, which can be calculated for each crack model according to the equations given in

Fig. 12. Time histories of mode I SIF K_I at the crack fronts of the fatigue critical connections: (a) connection CH-A05&06; (b) connection BW-05&06; (c) connection ST-A05 and (d) connection FW-15W8.

Section 3.1; and $\Delta\sigma_D$ is the constant amplitude fatigue limit of a detail category, prescribed in Eurocode 3 [2].

In the present study, several initial crack sizes $a_0 = 0.1, 0.15, 0.2$ and 0.25 mm were assumed respectively for the semi-elliptical surface cracks in connections BW-05&06 and ST-A05, and also for the through edge crack in connection CH-A05&06, so as to estimate the fatigue crack propagation life taking the initial crack size effects into account. Available inspection data for the aspect ratio a/c of semi-elliptical crack are very limited and also show large variations, as given in Table 2. As recommended by BS 7608 [31], an initial aspect ratio $a/c = 0.2$ was adopted for the semi-elliptical surface cracks, kept constant in crack propagation analysis for simplification. For the load carrying fillet weld connection FW-15W8, the incomplete penetration size equaling to the thickness of the gusset plate, 30 mm, should be included in the total crack size and thus, the initial half size of the central through crack was $(15 + a_0)$ in which a_0 was 0.1, 0.15, 0.2 and 0.25 mm as assumed.

Another important parameter for crack propagation analysis is the final critical crack size, which could be estimated by the fracture mechanics based criterion and the serviceability limit state of the structural

components. The critical crack size a_f at fracture for each welded connection given in Table 2 was determined by the LEFM K criterion as follows:

$$a_f = \frac{1}{\pi} \left(\frac{K_{mat}}{Y \cdot \sigma_{max}} \right)^2 \quad (26)$$

where, K_{mat} is the estimated material fracture toughness associated with the plate thickness; and σ_{max} is the maximum nominal stress produced by dead and live loads. Conservatively, a tensile residual stress $0.4f_y$ is included.

In addition, the crack size should not exceed the serviceability limit state of the structural components and generally, the thickness of the steel plate (in which the crack was detected or assumed) was considered as the maximum allowable crack size a_{max} , given in Table 2. Besides, the weld throat size was chosen as the maximum allowable crack extension size of the fillet weld root crack in connection FW-15W8. Lastly, the relative small crack size determined by the LEFM K criterion and the serviceability limit state was adopted as the final critical size, i.e. $a_{cr} = \min(a_f, a_{max})$.

Table 1
Comparisons between empirical formulas and global–local FE model in SIF calculations.

Critical connection	σ_{max} [MPa]	$a = 0.2$ mm			$a = 1.0$ mm			$a = 5.0$ mm		
		$^b K_{I,emp}$	$^b K_{I,FEM}$	c Error	$K_{I,emp}$	$K_{I,FEM}$	Error	$K_{I,emp}$	$K_{I,FEM}$	Error
CH-A05&06	6.87	35.9	33.4	+7.5%	52.6	48.9	+7.6%	77.2	71.9	+7.4%
BW-05&06	7.48	13.1	12.0	+9.2%	17.8	16.4	+8.5%	34.0	31.2	+9.0%
ST-A05	6.36	13.0	13.6	−4.4%	19.9	20.9	−4.8%	32.3	33.7	−4.2%
^a FW-15W8	3.03	23.6	25.6	−7.8%	25.3	27.8	−9.0%	38.3	42.8	−10.5%

^a The half crack size of the central through crack in the fillet weld connection FW-15W8 is $(a + 15)$ mm.

^b The unit of $K_{I,emp}$ and $K_{I,FEM}$ is in $\text{MPa} \cdot \text{mm}^{1/2}$.

^c The error between empirical solution and FE model calculation in SIF is defined as $\text{Error} = (K_{I,emp} - K_{I,FEM})/K_{I,FEM} \times 100\%$.

Table 2

Initial and critical sizes of the assumed cracks in the concerned welded connections.

Critical connection	Fatigue limit $\Delta\sigma_D$ [MPa]	Crack model	Initial crack size a_0 [mm]		Critical crack size a_{cr} [mm]		Aspect ratio a/c
			Literature	Fictitious	K criterion a_f	Serviceability a_{max}	
CH-A05&06	52.3	Through edge	0.1 [9]	0.011	11.6	20	—
BW-05&06	66.3	Semi-elliptical	0.2 [9]	0.035	19.5	25	0.01 [9]
ST-A05	59.0	Semi-elliptical	0.125 [36]	0.036	20.2	25	0.39 [36]
FW-15W8	26.5	Central through	Half size $(15 + a_0)$		20.2	22.7	—

4.2. Crack propagation analysis method

The remaining fatigue life of the cracked welded connection can be estimated through crack propagation analysis based on a crack growth relationship proposed by Paris and Erdogan [24], known as the Paris law given in Eq. (27):

$$\frac{da}{dN} = \begin{cases} C(\Delta K)^m & (\Delta K \geq \Delta K_{th}) \\ 0 & (\Delta K < \Delta K_{th}) \end{cases} \quad (27)$$

where, a is the crack size; N is the number of stress cycles; C and m are the material dependant crack growth parameters; ΔK_{th} is the threshold stress intensity factor range; and ΔK is the stress intensity factor range, which can be calculated by Eq. (28):

$$\Delta K = K_{max} - K_{min} = Y\Delta\sigma\sqrt{\pi a} \quad (28)$$

where, K_{max} and K_{min} are the maximum and minimum stress intensity factors corresponding to the maximum and minimum applied nominal stresses; $\Delta\sigma$ is the applied nominal stress range.

Integrating Eq. (27) from the initial crack size a_0 to the critical crack size a_{cr} and then substituting ΔK by Eq. (28), the fatigue crack propagation life in terms of the number of stress cycles can be obtained as:

$$N = \int_{a_0}^{a_{cr}} \frac{da}{C(\Delta K)^m} = \int_{a_0}^{a_{cr}} \frac{da}{C(Y\Delta\sigma\sqrt{\pi a})^m} \quad (29)$$

where, a_0 and a_{cr} are the initial and critical crack size, respectively.

In this study, the crack propagation analysis of the critical welded connection was performed in accordance with a flowchart as shown in Fig. 13. It includes two major stages, the equivalent stress range $\Delta\sigma_{eq}$ calculation and the crack growth simulation under constant amplitude stress range $\Delta\sigma_{eq}$. The variable amplitude stress history generated by a single train passage was treated as a typical stress block and the equivalent stress range $\Delta\sigma_{eq}$ was calculated based on the assumption of that, an equivalent crack extension was produced by a single stress cycle of $\Delta\sigma_{eq}$ and by several stress cycles due to one train passage. Subsequently, the crack propagation life of each critical welded connection, in terms of the number of train passages, was calculated under the constant amplitude stress range $\Delta\sigma_{eq}$.

As shown in Fig. 13, the detailed procedure to calculate the equivalent stress range $\Delta\sigma_{eq}$ was as follows:

- (1) The nominal stress history of the critical welded connection generated by a single train passage was converted into a stress spectrum (stress range $\Delta\sigma_i$ versus number of stress cycles n_i) by the Rainflow cycle counting method [22].
- (2) The low stress cycles, in which the SIF range ΔK_i was below the threshold SIF range ΔK_{th} , were cut off and an updated stress spectrum ($\Delta\sigma_j$ versus n_j) was obtained.
- (3) A small crack extension Δa_i within each stress cycle $\Delta\sigma_j$ was calculated cycle by cycle based on the Paris law and the total crack extension Δa and the updated crack size a_N were obtained.
- (4) Based on the equivalent crack extension assumption, the equivalent stress range $\Delta\sigma_{eq}$ was calculated:

$$\Delta\sigma_{eq} = \left[\Delta a / C \left(Y\sqrt{\pi a} \right)^m \right]^{1/m} \quad (30)$$

where, Δa is the total crack extension size during one train passage; $\bar{a}$ is the average crack size estimated as $\bar{a} = (a_0 + a_N)/2$.

The variable nominal stress histories applied on the critical welded connections CH-A05&06, BW-05&06, ST-A05 and FW-15W8 due to one passage of the high-speed train ETR500Y were used to calculate the corresponding equivalent stress range $\Delta\sigma_{eq}$. In each critical connection, the SIF range ΔK_i induced by any stress cycle of the loading history was below the threshold $\Delta K_{th} = 63 \text{ MPa} \cdot \text{mm}^{1/2}$, and no crack extension took place in any critical welded connection. Therefore, the estimated equivalent stress range $\Delta\sigma_{eq}$ was equal to zero. However, in this study, the threshold ΔK_{th} was not taken into account in calculations of the equivalent stress ranges, so as to evaluate the relative fatigue crack propagation life of the concerned welded connections.

Referring to Fig. 13, the fatigue crack growth calculation under the constant amplitude stress cycles (i.e. under the equivalent stress range $\Delta\sigma_{eq}$) was carried out as the following steps:

- (1) A small crack extension size $\Delta a_i = 0.001a_{i-1}$ was adopted and the current crack size $a_i = a_{i-1} + \Delta a_i$ was calculated.
- (2) The SIF ranges ΔK_{i-1} and ΔK_i associated with the crack size a_{i-1} and a_i were calculated under the equivalent stress range $\Delta\sigma_{eq}$.
- (3) If the difference between ΔK_i and ΔK_{i-1} was less than $0.001\Delta K_i$, i.e. $(\Delta K_i - \Delta K_{i-1}) < 0.001\Delta K_i$, computation was continued; otherwise, the crack extension should be modified as $\Delta a_i = \Delta a_i/2$ and computation was returned back to step (2).
- (4) The average crack growth rate was estimated by Eq. (31), and the number of cycles ΔN_i that needed to propagate the crack extension Δa_i was calculated by Eq. (32). The total number of cycles N_i corresponding to the crack size a_i was obtained as $N_i = N_i + \Delta N_i$ at the current step i .

$$(\overline{da/dN})_i = C(\overline{\Delta K}_i)^m \quad (31)$$

where, $\overline{\Delta K}_i$ is the mean value of ΔK_i and ΔK_{i-1} .

$$\Delta N_i = \Delta a_i / (\overline{da/dN})_i \quad (32)$$

- (5) Steps (1) to (4) were repeated until the crack size a_i exceeded the critical size a_{cr} . Finally, the crack size versus the number of cycles curve $a_i - N_i$ could be obtained.

5. Results and discussion

According to the flowchart given in Fig. 13, the algorithms for equivalent stress range calculation and crack propagation analysis were programmed in MATLAB [37]. The equivalent stress range $\Delta\sigma_{eq}$ of each critical connection was calculated based on the variable nominal stress history induced by a single train passage and the results are shown in

Fig. 13. Flowchart of fatigue crack propagation analysis.

Table 3. Then, the fatigue crack propagation under the constant stress range $\Delta\sigma_{eq}$ was carried out and the crack growth curves (i.e. crack size a versus number of train passages N_{tr}) with different initial crack sizes a_0 are shown in Fig. 14. With the crack growth curve, the fatigue life during which the crack propagated from the initial size a_0 to the critical size a_{cr} was obtained for each critical welded connection as listed in Table 3.

As shown in Fig. 14, the fatigue crack propagation life decreases as the initial crack size increases. Taking connection ST-A05 shown in Fig. 14c as an example, the fatigue propagation life represented by N_{tr} reduces approximately 20% when the initial crack size increases from 0.10 mm to 0.25 mm. It is found in Fig. 14 that the crack propagates

slowly at the early stage which forms the major part of the propagation life, while it grows faster at the later period. It also indicates that the final critical crack size has less effect on the fatigue propagation life as compared to the initial crack size. Actually, it is more appropriate to choose the crack size when it is accelerating as the allowable critical size, so that enough time is available for repair, retrofitting or replacement. For connections BW-05&06 and ST-A05 as shown in Fig. 14b and c, fatigue crack repair and reinforcement are recommended when the crack size grows to 5 mm although the critical size estimated by the LEFM K criterion is about 20 mm.

Table 3

Fatigue crack propagation analysis results of the critical welded connections.

Critical connection	$\Delta\sigma_{eq}$ [MPa]	a_{cr} [mm]	Number of train passages N_{tr} when crack propagates from a_0 to a_{cr}			
			$a_0 = 0.10$ mm	$a_0 = 0.15$ mm	$a_0 = 0.20$ mm	$a_0 = 0.25$ mm
CH-A05&06	7.79	11.6	5.218E08	4.874E08	4.618E08	4.415E08
BW-05&06	8.40	19.5	2.035E10	1.891E10	1.779E10	1.685E10
ST-A05	7.28	20.2	1.765E10	1.606E10	1.493E10	1.405E10
^a FW-15W8	3.69	20.2	4.093E09	4.007E09	3.923E09	3.841E09

^a The initial half crack size of the connection FW-15W8 is $(15 + a_0)$ mm.

Fig. 14. Fatigue crack growth curves of the critical welded connections: (a) connection CH-A05&06; (b) connection BW-05&06; (c) connection ST-A05 and (d) connection FW-15W8.

The fatigue propagation life of the studied critical welded connections with varying initial crack sizes is clearly shown in Fig. 15. The cope hole detail CH-A05&06 and the load carrying fillet weld connection FW-15W8 have a relatively shorter fatigue propagation life compared with the assembling butt weld connection BW-05&06 and the transverse stiffener fillet welded detail ST-A05.

As shown in Fig. 16, the fatigue life of the four critical welded connections predicted by the LEFM approach in the present study is compared with that estimated by the traditional *S-N* method (according to Eurocode 3 [2]) performed in the previous research of the authors [1]. In the previous study, fatigue damage *D* due to one passage of the high-speed train was presented and fatigue life is calculated as the reciprocal of *D* on the basis of Palmgren–Miner linear damage rule. In general, fatigue life of each critical welded connection estimated by the traditional *S-N* method is relatively larger than that predicted by the

LEFM approach, which seems reasonable as the *S-N* method takes account of the total fatigue life including crack initiation and propagation. However, a similar tendency is predicted by both methods that the fatigue life of connections CH-A05&06 and FW-15W8 is relatively shorter than that of connections BW-05&06 and ST-A05.

6. Conclusions

In this study, an enhanced fatigue evaluation for a steel–concrete composite railway bridge, the Sesia viaduct, was carried out after a preliminary fatigue assessment based on the traditional *S-N* method in the previous work [1]. Fatigue crack propagation analyses based on the Paris law and LEFM were performed on the critical welded connections, which were identified by the preliminary fatigue assessment. The following conclusions can be drawn from the present study:

Fig. 15. Fatigue crack propagation life of the critical welded connection with varying initial crack sizes.

Fig. 16. Fatigue life evaluated by the traditional *S-N* method and the LEFM approach.

- (1) Initial cracks were introduced into the identified critical welded connections CH-A05&06, BW-05&06, ST-A05 and FW-15W8, and the refined local FE models with assumed cracks were incorporated into the global FE model of the bridge span. The applicability and the accuracy of the empirical SIF formulas for the critical welded connections were validated by dynamic analyses using the global–local FE model with three-dimensional cracks.
- (2) The variable amplitude stress history sustained by each cracked connection due to one passage of the high-speed train ETR500Y was converted into one stress cycle with an equivalent stress range $\Delta\sigma_{eq}$, based on the proposed equivalent crack extension assumption. The fatigue crack propagation under constant amplitude stress range $\Delta\sigma_{eq}$ was analyzed based on the Paris law and LEFM. The crack growth curve of each critical connection, expressed by crack size versus number of train passages, was obtained and the fatigue propagation life was predicted. The initial crack size has a significant effect on the fatigue propagation life, while the final critical crack size is less important.
- (3) In fatigue assessment of the four concerned critical welded connections, fatigue life predicted by the LEFM approach in the present study is shorter than that evaluated by the *S-N* method in the previous research [1]. Fatigue life of connections CH-A05&06 and FW-15W8 is relatively shorter than that of the connections BW-05&06 and ST-A05, as estimated by both methods. The LEFM provides an alternative and a more general approach to evaluate the fatigue life of welded details, especially when the traditional *S-N* method is inapplicable.

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