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Abstract: This paper presents an overview of recent research efforts by the authors and co-workers on the fatigue assessment of old metallic railway bridges. The investigation focuses on the behaviour of riveted stringer-to-cross-girder connections in a typical, short-span bridge. A generic methodology, which is based on nominal stresses and the S-N method, is presented first, followed by a more detailed analysis using a recently developed fatigue assessment theory, which is based on local stress distributions. The discussion is made within a deterministic as well as a probabilistic context and typical results are presented in terms of fatigue damage and remaining fatigue life.

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To whom it may concern:

Please find enclosed the electronic form of “**Fatigue evaluation of riveted railway bridges through global and local analysis**” by Boulent Imam and Timothy Righiniotis. We would like to have this manuscript reviewed by the **Journal of Constructional Steel Research**.

Should you need to contact me, please use the above address or call me at +44 (1483) 689679. You may also contact me by fax at +44 (1483) 682135 or via e-mail at b.imam@surrey.ac.uk.

Sincerely,

Boulent Imam

Fatigue evaluation of riveted railway bridges through global and local analysis

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Abstract

This paper presents an overview of recent research efforts by the authors and co-workers on the fatigue assessment of old metallic railway bridges. The investigation focuses on the behaviour of riveted stringer-to-cross-girder connections in a typical, short-span bridge. A generic methodology, which is based on nominal stresses and the S-N method, is presented first, followed by a more detailed analysis using a recently developed fatigue assessment theory, which is based on local stress distributions. The discussion is made within a deterministic as well as a probabilistic context and typical results are presented in terms of fatigue damage and remaining fatigue life.

Keywords: Riveted bridges; remaining fatigue life; S-N method; finite element method; nominal stresses; local stresses.

1. Introduction

A significant portion of the railway bridge infrastructure in the UK and parts of Europe and North America is of wrought iron and old steel construction and in many cases already exceeds 100 years of age. These bridges, which are constructed using rivets,

1 were built before standardisation and widespread use of design codes at a time when
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3 fatigue was certainly not an issue that was taken into account in design. Having
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5 experienced continuously increasing axle loads and train frequencies during their
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7 service life, questions regarding their fatigue performance have been raised. However,
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9 so far, these bridges seem able to cope with current load demands with relatively few
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11 cases of fatigue damage being reported.
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18 Due to the large number of old steel and wrought iron railway bridges in UK (it is
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20 estimated that there are more than 6000), replacement of all these structures will be
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22 extremely expensive and practically impossible unless phased-in over several decades.
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24 In many cases, repair of these old bridges may be sufficient and certainly more
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26 economic. However, even this course of action is likely to create logistical problems on
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28 the railway network, if deemed necessary on a large scale. A better understanding of a
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30 bridge's condition will undoubtedly lead to a more effective use of resources and can
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32 reduce the number of repairs.
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40 An overview of recent research efforts by the authors and co-workers on the fatigue
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42 assessment of old metallic railway bridges is presented in this paper. The current
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44 practice for assessing old metallic bridges for fatigue is based on the traditional S-N
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46 method (BS 5400 [1], Railtrack [2], Eurocode [3]). The S-N method relies on assigning,
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48 depending on geometry, loading, etc, a specific class to the bridge detail in question.
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50 The fatigue damage is then estimated based on the nominal stress applied to the detail
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52 and its corresponding, code-specified S-N curve. However, only a limited number of
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54 details associated with older bridges are included in the relevant bridge standards and
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1 there are many cases of actual bridge details that cannot be clearly classified or may
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3 even not be classified at all. One such example is the case of riveted stringer-to-cross-
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5 girder connections, which, based on the research carried out on riveted bridges over the
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7 last two decades [4-7] and the present study [8], were identified as fatigue-critical
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9 details. Stringer-to-cross-girder connections are associated with complex loading and
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11 secondary, deformation-induced effects and their assessment based on the S-N method
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13 can lead to uncertain results which can make difficult the correct condition assessment
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15 and planning of maintenance strategies.
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23 In recent years, as an alternative to the traditional S-N method, new fatigue assessment
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25 methodologies which are based on local stresses, rather than nominal stresses, have
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27 been developed [9-11]. These are based on linear elastic finite element analyses of the
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29 details in question and can be employed for the fatigue assessment of complex details,
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31 for which the S-N method is not straightforward to apply. The most widely used such
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33 methods are the ‘hot-spot method [9]’ and the ‘Theory of Critical Distances (TCD)
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35 [11]’. The former has been widely used in the offshore industry in conjunction with
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37 welded details, whereas the latter has been successfully employed in the case of
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39 automotive and mechanical components. Both of these methods need to be developed
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41 and adapted for bridge application.
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49 The authors have employed both the S-N method and the TCD for the fatigue
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51 assessment of stringer-to-cross-girder connections in old metallic railway bridges and
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53 the development of a generic fatigue assessment methodology. A flowchart of the
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55 proposed methodology is presented in Fig. 1. This paper presents typical results
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obtained from both methods as well as comparisons between them. The first part of the paper deals with fatigue analysis using the S-N method which is termed ‘global finite element analysis’. Consequently, emphasis is given to the S-N classification of fatigue-critical riveted details. A historical rail traffic model is specifically developed [12] in order to estimate the fraction of the bridge’s fatigue life that has already been expended. As shown in Fig. 1, train loading is first converted into nominal stress histories by using a finite element (FE) model of a typical, short-span, plate girder, riveted railway bridge. Fatigue damage is then calculated by using the resulting stress histories and the bridge connections are ranked according to their relative criticality. Probabilistic methods are introduced at this stage, allowing for loading, material and modelling uncertainties to be accounted for. The results obtained from the probabilistic analysis, which are presented in terms of time-dependant failure probabilities, are then compared with specified target reliabilities and interpreted within the context of the large number of connections present in the railway bridge network.

The global FE analysis is followed by a refined global-local FE analysis of the entire bridge under train loading, as can be seen in Fig. 1. The local model takes into account all the geometrical features of the critical connection identified during the global analysis and, therefore, aims at fully capturing the behaviour of the connection in terms of its complex loading and secondary effects. This global-local analysis aims at identifying the relative fatigue criticality of the different connection components (angle fillet, rivet holes, rivets) and estimating their remaining fatigue life. This is carried out by considering the local stress distribution in the vicinity of the investigated stress concentrations within the context of the TCD [11], thus identifying potential crack

initiation sites within the connection. The global-local FE analysis can also be set within a probabilistic format as shown in Fig. 1. The results obtained from this stage could serve as input to further crack growth analyses and assist in inspection planning and repairs.

2. Rail traffic models

A key aspect in determining the remaining fatigue life of old metallic bridges is the calculation of the portion of the fatigue life that has already been expended. This, in turn, requires knowledge of the traffic history of the bridge both in terms of the different types of trains that have crossed the bridge as well as their axle weights and corresponding frequencies. BS 5400 suggests the use of standard load spectra for fatigue damage calculation, which are derived in terms of three levels of traffic: light, medium or heavy traffic, each consisting of a number of suggested typical trains with corresponding annual frequencies [1]. Light traffic consists of passenger and suburban trains, whereas heavy traffic is solely composed of freight trains. Both passenger and freight trains are included in medium traffic. These standard traffic types are deemed representative of current rail traffic in the UK. Similar traffic models are also suggested in Eurocode for European rail lines [13].

However, the type and frequency of the loading experienced by railway bridges in the past is different from those experienced today. For example, in the past, the heaviest part of the train was the locomotive which was usually followed by a number of lighter wagons. Today, the wagons following the locomotive constitute the heaviest train component, especially in the case of freight trains. No details regarding the load history of old metallic bridges could be found in UK standards. However, the International

1 Union of Railways (UIC) provides examples of representative past traffic loads for
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3 European rail lines [14]. Past loading can be estimated using historical traffic records
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5 but often this is very time-consuming. In many cases, simplifications may produce
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7 realistic traffic models which can capture the conditions that prevailed on the railway
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9 network over the last century. A simplified historical traffic model, based on
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11 information gleaned from Network Rail records is presented in Table 1 [12]. As can be
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13 seen, the traffic model is divided into four distinct periods, each associated with
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15 particular characteristics in rail traffic. For the last period (1970-2007), the BS 5400
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17 medium traffic model is adopted [1]. Further details regarding the proposed model and
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19 the characteristics of the historical trains such as locomotive and wagon dimensions and
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21 axle spacings can be found in [8] and [12].
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30 For remaining fatigue life estimates, prediction of live load evolution over the next few
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32 decades may also be important. For example, load evolution in highway bridges has
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34 been found to affect fatigue life estimates for welded connections considerably [15].
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36 Since rail traffic is generally less uncertain than highway traffic, load evolution
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38 assumptions are usually more straightforward for the former. For railway bridges,
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40 various scenarios such as increase in train frequencies, increase in train axle loads,
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42 increase in train lengths, or different combinations of these can be considered in order to
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44 meet future target increases in passenger and freight traffic. Each of these scenarios can
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46 be expected to have a different effect on remaining fatigue life estimates and the failure
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48 probabilities of the bridge connections.
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3. Global finite element analysis

3.1. Bridge model

The fatigue assessment methodology shown in Fig. 1 was developed for typical, short span, riveted railway bridges such as the one shown in Fig. 2(a) whereas Fig. 2(b) shows the corresponding global FE model together with relevant dimensions and the connection clarification. The bridge FE model was developed using the commercial FE-package ABAQUS [16]. The superstructure of the twin-track unballasted bridge consists of three riveted, built-up main plate girders and four longitudinal rows of built-up stringers interconnected with built-up cross-girders. For the purposes of the global FE analysis, all built-up members are transformed into equivalent I-sections having the same depth and the same second moment of area. The bridge is assumed to be simply supported at the ends of the three main girders. All the members are modelled using 8-noded shell elements and elastic behaviour is assumed using a Young's modulus of 200 GPa and a Poisson's ratio of 0.3. The former is a typical value for UK wrought iron [17]. The train axle loads are applied directly to the top flange of the stringers thus neglecting the beneficial effect of load spread due to the rails and sleepers.

Two types of connections can be identified in this type of bridge; namely, stringer-to-cross-girder (S) and cross-girder-to-main girder (C) connections (see Fig. 2(b)). Such connections are made up of angle cleats riveted to the webs of the members and are usually assumed to behave as pinned connections, able to transfer shear forces only. This is justifiable for an ultimate limit state assessment since this assumption results in the maximum bending moment in the midspan of the member. However, fatigue tests carried out on this type of bridge connections have revealed that they are able to develop considerable amounts of bending moment [18,19], which is important from a

1 fatigue standpoint. Furthermore, comparisons of stress histories obtained by using
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3 analytical/numerical bridge models with field measurements have confirmed that the
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5 assumption of fully-fixed connections results in more realistic stress histories [20,21].
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7 Accordingly, in previous studies [8,12], the various bridge members were tied to each
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9 other in the global FE model, a modelling assumption which is equivalent to the
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11 assumption of fixed connections. Parametric studies, using this model in order to
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13 ascertain the effect of connection fixity on remaining fatigue life estimates, have
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15 revealed that the fully-fixed assumption results in the highest fatigue damage
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17 accumulation, and hence conservative fatigue life estimation for stringer-to-cross-girder
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19 connections [22].
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27 *3.2. Fatigue detail classification*

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29 In order to calculate fatigue damage using the S-N method, the different detail
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31 classifications that are available for riveted bridge connections are presented first. BS
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33 5400 [1] suggests the use of its Class D for the fatigue assessment of lapped or spliced
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35 riveted connections. Furthermore, Class B can also be used for riveted details [23]. This
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37 class relates to the stress at the edge of a hole and, therefore, its S-N curve has to be
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39 divided by a stress concentration factor of 2.4 (suggested in BS 5400 [1] for holes) in
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41 order to use it in conjunction with nominal stresses. This class is hereafter referred to as
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43 modified Class B. The UK railway assessment code [2] makes a distinction between
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45 steel and wrought iron elements and proposes a class for the fatigue assessment of
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47 wrought iron riveted details (Class WI-rivet). It has to be mentioned that Xie et al. [23]
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49 suggested a different S-N curve for the fatigue assessment of riveted flanges. This was
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1 based on the results of experiments on full-scale riveted girders that were carried out by
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3 the authors coupled with experimental results obtained from the literature.
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8 The mean S-N curve for Class D and the mean and design S-N curves for modified
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10 Class B are plotted in Fig. 3 together with all the full-scale experimental results
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12 retrieved from the literature for old steel and wrought iron riveted members and
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14 connections [8]. The design S-N curve suggested by Xie et al. [23], which is effectively
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16 identical with the modified Class B mean S-N curve, is also shown. Fig. 4 presents
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18 experimental data from fatigue tests on wrought iron riveted elements together with the
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20 mean and design S-N curve for Class WI-rivet. It should be noted that the experimental
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22 results shown in Fig. 3 relate to full-scale fatigue tests of both old steel and wrought
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24 iron riveted girders. On the other hand, due to the limited number of test results on
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26 wrought iron elements, both full-scale and small-scale test results on wrought iron
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28 members and connections are shown in Fig. 4.
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37 A considerable degree of scatter can be observed in the fatigue test results on riveted
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39 members and connections presented in Fig. 3 and 4. The data presented in Fig. 3 and 4
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41 imply a higher standard deviation than that associated with the modified Class B, Class
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43 D or Class WI-rivet. Moreover, the scatter seen in Fig. 3 and 4 is greater than that
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45 evidenced with welded details. Several possible reasons contributing to this scatter may
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47 be identified as being the wide range of rivet clamping force values in the specimens,
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49 the different stress ratios that were applied during different tests, the different material
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51 properties (old steel versus wrought iron), the method of hole preparation (punched
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53 versus drilled), the possible presence of corrosion on the test specimens, the variation in
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1 damage accumulated before removing the specimens for testing and, finally, the
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3 different termination criteria used in the tests (first observed cracking, fracture or
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5 excessive deformation) [8] .
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10 As can be seen in Fig. 3, the Class D mean S-N curve lies rather close to the upper
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12 bound of the experimental data and, for this reason, was not considered further. The
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14 design S-N curve proposed by Xie et al. can also be seen to be optimistic with respect to
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16 several experimental results. On the other hand, the modified Class B design S-N curve
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18 can be seen to capture well the lower bound of test results, although its corresponding
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20 mean curve is somewhat conservative with respect to the test results. From Fig. 4, it can
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22 be seen that the Class WI-rivet curve captures the scatter for wrought iron riveted
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24 connections reasonably well.
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32 It has to be emphasised that there is very limited published work on full-scale tests of
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34 riveted bridge connections such as stringer-to-cross-girder connections [8]. Due to the
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36 complex behaviour of such connections, which involves out-of-plane deformations and
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38 the development of secondary stresses [19], it is often difficult to express these test
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40 results on an S-N basis using nominal stresses. As is well known, the S-N curves found
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42 in fatigue codes are generally developed for uniaxial stress conditions and it has yet to
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44 be verified whether they can be used confidently for estimating the fatigue life of such
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46 riveted connections. Based on the results presented in Fig. 3 and 4 and in view of the
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48 lack of specific test results on stringer-to-cross-girder connections, the modified Class B
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50 and Class WI-rivet curves are tentatively proposed for fatigue damage calculation for
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52 this type of connections.
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3.3. *Fatigue damage calculation*

The S-N curves related to riveted details shown in Fig. 3 and 4 are plotted in terms of nominal stresses. Therefore, stress history outputs for fatigue damage calculations have to be free of stress concentration effects since these are implicitly taken into account through the S-N curves. In previous work [12], stress concentration effects at the location of the bridge connections were found to diminish at a distance of 250 mm from the stringer-to-cross-girder interfaces. As a result, by traversing the trains of the rail traffic model (Table 1) over the bridge, stress history outputs are obtained at that distance from the connections. Dynamic Amplification Factors (DAF), given in various structural codes and published field measurements, are then used in a multiplicative fashion to modify the statically calculated stress histories. A more detailed discussion on the DAF can be found in [12] and [24]. The stress histories are first converted into stress range blocks using the rainflow counting method. Miner's rule [25] is then applied in order to calculate the total fatigue damage of each connection in the period 1900-2007 and rank them according to their fatigue criticality. For deterministic calculations, fatigue damage is calculated by using the mean minus two standard deviations, two-slope (design) S-N curves (modified Class B and Class WI-rivet) which correspond to a 2.3% probability of failure.

3.4. *Probabilistic approach*

The deterministic fatigue analysis, which is carried out in order to rank the riveted bridge connections according to their S-N calculated fatigue damage, can be extended within a probabilistic framework, as shown in Fig. 1. A probabilistic approach is, in principle, more appropriate for fatigue assessment due to the uncertainties associated

with fatigue loading, material resistance and modelling. For the purposes of this analysis, fatigue loading is randomised through the annual train frequencies and the dynamic amplification of the statically calculated stress histories [26,27]. On the material side, the S-N curves pertaining to the different fatigue classifications (see Fig. 3 and 4) and the cumulative damage model are treated probabilistically [28]. Lastly, modelling uncertainty is represented by the differences between calculated and actual stresses. Since calculated stresses (analytically or numerically) are often higher than their actual counterparts obtained through field measurements, a random factor is used to reduce the former [28]. Monte Carlo (MC) simulation is first used to generate stress ranges resulting from random train bridge crossings and to incorporate DAF and model uncertainties. The resulting probabilistic annual response spectra are then used through further MC simulation to estimate the reliability profiles of the connections. The results obtained from the probabilistic analysis, which are in terms of probability of failure versus time, are particularly useful when viewed within the context of the large number of riveted connections in the bridge network. Further details regarding the random models used for the probabilistic analysis can be found in [8] and [28].

3.5. *Results and discussion*

The global FE analysis, which forms the first step of the fatigue assessment methodology (see Fig. 1), aims at identifying the most critical connections on the bridge. This is achieved by calculating the S-N based fatigue damage of all the connections under the rail traffic model presented in Table 1 and ranking them accordingly. Ignoring any dynamic amplification, the fatigue ranking of the stringer-to-cross-girder connections with respect to the total fatigue damage, calculated in the

period 1900-2007 and for the two detail classifications (modified Class B and Class WI-rivet), is shown in Fig. 5 [24]. As can be seen, the fatigue-critical connections are the inner stringer-to-cross-girder connections adjacent to the midspan region of the bridge. The fatigue damage of the cross-girder-to-main girder connections is found to be considerably lower [24].

The damage evolution of the most fatigue-critical connection from the year 1900 up to 2040 is shown in Fig. 6. Two set of results are presented i.e. with and without considering dynamic amplification. In the former case, dynamic amplification factors were obtained from the UK railway assessment code [2]. Once again, design S-N curves are used for damage calculation. In obtaining these results, a load evolution scenario involving a 3% annual increase in all train frequencies from the year 2005 until 2010 and the introduction of 30 ton axles in freight wagons thereafter was assumed [29]. It is evident that the fatigue damage under historical train loads (period 1900-1970) is quite low. However, beyond 1970, there is a notable increase in the damage accumulation rate of the connections and this is due to the introduction of heavier freight trains with 25 ton axles. There is a further increase in the damage accumulation rate with the assumed introduction of 30 ton axles in the freight train wagons in 2010, which is expected to reduce significantly the remaining fatigue life of the bridge connections. It can also be seen in Fig. 6 that there is a considerable increase in the damage of the connections due to dynamic amplification.

Further deterministic studies carried out within this first step of the proposed fatigue assessment methodology have also led to some important observations which can be summarised as follows [8]:

- Only few stress cycles are above the fatigue limits of the detail classifications considered.
- The behaviour of the stringer-to-cross-girder connections is axle dominated.
- The mean stress range experienced by the connection is doubled in value between the beginning and the end of the 20th century.
- Although the simultaneous passage of two trains over the bridge results in a considerable increase in the stress ranges experienced by the critical connections, it has a negligible influence on their fatigue life, given that it constitutes a rare event.
- The effect of second track loading (i.e. left track in Fig. 2(b)) on the fatigue damage of the connections of the first track (i.e. right track in Fig. 2(b)) is negligible.

Typical results obtained from the probabilistic analysis [28] are shown in Fig. 7 which depicts the probability of failure of the most fatigue-critical connection versus time for the two different detail classifications (modified Class B and Class WI-rivet). Results pertaining to the load evolution scenario mentioned previously are also shown for a WI-rivet classification. As mentioned earlier, these results incorporate randomness in loading, resistance and modelling. The value of the mean remaining fatigue life and its standard deviation are also shown in the figure. It is important to emphasise that, from an engineering point of view, the significance of these distributions lies in the very high standard deviations rather than in the mean values. The estimated values for the

standard deviations offer a quantitative assessment of the overall uncertainty associated with fatigue evaluation procedures and underline the importance of adopting intensified inspection and management plans for this class of bridges. Given the large number of similar stringer-to-cross-girder connections in the railway bridge network (it is estimated roughly that almost 100000 similar connections are in-service today [8]), target reliabilities that are considerably lower than those pertaining to a 97.7% (design) probability of survival should generally be considered during a fatigue assessment. For example, with a target failure probability of 10^{-3} (100 connection failures) the remaining fatigue life is between 40-80 years. It is also evident in Fig. 7 that fatigue life estimates are very sensitive to detail classification. Consideration of load evolution can be seen to affect fatigue life estimates considerably, with failure probabilities being at least an order of magnitude greater than their no-evolution counterparts [29].

Further probabilistic studies have revealed some interesting points that can be summarised as follows [8,28]:

- The annual response spectra developed for each period of the rail traffic model can be approximated reasonably well through Weibull distributions.
- Remaining fatigue lives are approximately lognormally distributed.
- Model uncertainties (e.g. differences between measured and calculated stresses) play a significant role for fatigue life estimates.
- Uncertainties in the cumulative damage model, dynamic amplification and the frequency of train traffic are comparatively of less importance.

- In terms of load evolution, the introduction of higher axle loads is expected to lead to a greater reduction in the remaining fatigue life of old metallic bridges when compared to the option of increasing the train frequencies only [29].

The third point above highlights the importance of field monitoring for old bridges approaching the end of their useful life in order to provide reliable fatigue life estimates. Field measurements in riveted railway bridges would be highly beneficial since they would provide means of calibrating and verifying the analytical/numerical bridge models.

4. Global-local finite element analysis

4.1. Overview

The second main step of the fatigue assessment methodology, which is referred to as the global-local FE analysis in the flowchart of Fig. 1, involves a more detailed stress analysis of the fatigue-critical riveted bridge connection identified from the global analysis. For this purpose, the global bridge model is enhanced at the location of the most fatigue-critical connection by introducing the detailed connection geometry. At this stage, focus is given to the local stress distributions in the vicinity of the stress concentrations, as opposed to the traditionally used nominal stresses considered in the global model. Moreover, instead of using detail-specific S-N curves, which implicitly take into account stress concentration effects, the plain material S-N behaviour is considered here. This refined analysis is able to capture secondary, deformation induced effects, which are very difficult to account for and often overlooked in a standard S-N treatment such as the one presented earlier in section 3.

4.2. *Connection model*

A close-up view of the global-local model at the most critical connection location is depicted in Fig. 8. This particular connection consists of four $76 \times 76 \times 12.6$ mm angle cleats, each riveted to the stringer and cross-girder webs using two and three 19 mm rivets, respectively. The connection components (angles, rivets) and part of the stringers and the cross-girder are now modelled with brick elements using the commercial FE-package ABAQUS [16]. Contact and friction between the individual parts of the connection as well as the rivet clamping force are introduced in the FE model. In contrast to the global bridge model of Fig. 2(b), where a simplified representation of the riveted connections is used, the global-local model permits investigation of the fatigue damage of the individual elements of the connection (angles, rivets). This model also allows the local flexibility of the connection and secondary effects to be accounted for.

4.3. *The Theory of Critical Distances*

A number of new methods which rely on local stresses have been recently developed for the fatigue assessment of connections [9-11]. These methods can be used in conjunction with linear elastic FE analysis in order to estimate the fatigue damage of complex details in cases where it is difficult to define a characteristic ('nominal') stress value or to classify the detail according to standards. The hot-spot method [9] and the structural stress method [10] have been primarily developed and verified for welded details. On the other hand, the TCD [11,30] has been shown to be successful in predicting the constant amplitude fatigue limit of a wide range of details ranging from welded connections [31,32] to mechanical components [33-35] as well as in predicting fatigue damage under variable amplitude loading [34]. It has also been shown to be applicable

1 to a wide range of metals ranging from cast iron [33,35] to steel [30,33,34] and
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3 aluminium [33].
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8 Within the framework of the methodology presented in this paper, the TCD is employed
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10 in order to estimate the remaining fatigue life of the different components of the
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12 connection [36]. This theory requires the calculation of a ‘critical distance’ associated
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14 with a volume in the vicinity of the notch tip, within which stresses are to be averaged.
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16 This average stress is then considered in combination with the plain material S-N curve
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18 and Miner’s rule to estimate the fatigue damage of the connection. The critical distance
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20 is a function of material properties, e.g. the crack propagation threshold and the fatigue
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22 limit [11,30], both of which can be obtained through fatigue tests on plain material
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24 specimens. More details regarding the TCD can be found in [36], where its robustness
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26 in predicting the fatigue life of a stringer-to-cross-girder connection has been
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28 established.
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37 *4.4. Fatigue damage calculation*

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39 By traversing the BS 5400 medium traffic trains [1] over the bridge, histories of the
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41 maximum principal stress are obtained within all the elements of the critical volume of
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43 the stress concentration under investigation (angle holes, fillet, rivets). These stresses
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45 are then averaged within the critical volume, for each step of the analysis, and the stress
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47 history of the average stress is then converted into stress range blocks through rainflow
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49 counting [36]. Plain material S-N curves are used for fatigue damage calculations. By
50
51 applying Miner’s rule to the stress range blocks, the connection components are ranked
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53 according to their fatigue criticality and fatigue crack initiation hot spots are identified.
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1 Considering the very limited number of experimental data on plain wrought-iron
2 material near the fatigue limit, a single slope S-N curve (slope 1/5) with a fatigue limit
3 of 183 MPa at 2×10^6 cycles is used for damage calculations. This S-N curve was
4 established in [37], following a series of fatigue tests on plain (un-notched) wrought
5 iron specimens at a load ratio of 0.1. With regard to the crack propagation threshold, a
6 value of $13.5 \text{ MPa} \cdot \text{m}^{1/2}$ is assumed. This value was suggested in [38] following a series
7 of crack growth experiments on wrought iron at a load ratio of 0.1.
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10 11 12 13 14 15 16 17 18 19 20 21 *4.5. Probabilistic approach and system effects*

22 In the same way as in the global analysis, the deterministic results obtained from the
23 global-local analysis can be extended within a probabilistic framework by taking into
24 account uncertainties due to fatigue loading, material resistance and modelling [39]. The
25 probabilistic global-local analysis is carried out by considering the same random
26 variables as in the global analysis. Annual response spectra are generated for each hot-
27 spot, which are then combined with the resistance uncertainties in a reliability-based
28 formulation in order to obtain failure probability versus time profiles and remaining
29 fatigue life estimates for each hot-spot on each individual connection component.
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45 It is important to note that riveted connections are made up of a number of different
46 components i.e. angle cleats and rivets and, therefore, failure of an individual
47 component may not lead to total connection failure. For example, the stringer-to-cross-
48 girder connection shown in Fig. 8 consists of 4 angle cleats and 10 rivets. To estimate
49 the reliability of the connection, the problem can be extended beyond the level of
50 individual components failure and can be investigated from a systems point of view. For
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1 this purpose, the connection can be considered as a system consisting of 4 sub-systems,
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3 which are, in this case, the angle cleats. Each angle cleat, in turn, consists of various
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5 hot-spots as its individual elements (e.g. holes, rivets, fillets). Through different
6
7 assumptions regarding the system representation, the probability of fatigue failure of the
8
9 connection can be estimated. Further details regarding the probabilistic global-local
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11 analysis and the system approach can be found in [39].
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18 *4.6. Results and discussion*

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20 The stress analysis of the global-local model reveals that the passage of trains over the
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22 bridge subjects the stringer-to-cross-girder connections to out-of-plane deformations
23
24 [8]. Although these are very small, they result in high secondary stresses around the
25
26 rivet holes, the angle fillet and at the rivet head-to-shank junctions which may result in
27
28 fatigue cracking of the connection. Fig. 9 depicts the total fatigue damage of different
29
30 hot spots on different components of the connection calculated for the period 1970-
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32 2007, for two different rivet clamping forces (100 and 200 MPa) [36]. Remaining
33
34 fatigue life estimates, assuming no load evolution in the future, are also shown. The
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36 results are presented in terms of the ratio of the damage predicted from the global-local
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38 model (using TCD) to the damage predicted by the global model and the traditional S-N
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40 approach (Fig. 5). The expected beneficial effect of high clamping force in reducing
41
42 fatigue damage is evident in the figure. An exception is the angle fillet, which becomes
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44 fatigue-critical for the higher rivet clamping force. Overall, Fig. 9 demonstrates that
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46 hole 5 on the stringer part of the connection (see Fig. 8) is the most highly damaged part
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48 of the connection irrespective of the level of clamping. Furthermore, as a result of out-
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50 of-plane deformations, rivet 3 on the cross-girder part of the connection (see Fig. 8)
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appears to be the next most highly damaged component of the connection, for the case of the lower clamping stress.

It can also be seen in Fig. 9 that the global-local model results in higher damage estimates for the most critical regions of the connection (hole 5, rivet 3 and angle fillet), which, depending on the clamping stress, can be by a factor of 3.5. In terms of these critical regions, the differences with the global model are reduced for higher clamping stresses.

The effect of several rivet defects scenarios, typically encountered in riveted railway bridges, on the fatigue damage of the connection has also been investigated through the global-local model [8]. These scenarios include the loss of clamping force in the top rivet connecting the angle to the cross-girder web, the loss of the entire rivet, smaller rivet head, offset rivet head and the presence of clearance between the rivet and its corresponding hole. It was found that defects in any one of the rivets influence mostly its adjacent locations on the connection. The most damaging scenarios were found to be the presence of clearance between the rivet shank and rivet hole and the loss of a rivet [8].

Typical results obtained from the probabilistic global-local analysis [39] are shown in Fig. 10 and 11. Fig. 10 depicts the probability of fatigue failure from year 2008 onwards for the different connection hot-spots whereas in Fig. 11, the system failure probabilities, assuming that the connection is represented by one-angle, two-angle, and four-angle systems are shown together with the results obtained from the probabilistic

1 global S-N analysis. The results are presented for a 200 MPa clamping force in all
2 rivets. It is evident from Fig. 10 that there is a very wide range in the probabilities of
3 failure of the different hot-spots. The remaining fatigue lives of the different hot-spots
4 range, for a 2.3% (design) probability of failure, from a couple of years up to more than
5 100 years. This illustrates the localised nature of the fatigue phenomenon with the
6 damage being initiated from specific stress concentrations within a connection. In
7 agreement with the results obtained from the deterministic global-local analysis (Fig. 9),
8 the highest failure probabilities (lowest remaining lives) are obtained for hole 5, angle
9 fillet and rivet 3.

10 The system reliability profiles in Fig. 11 show that the remaining life of the connection
11 is sensitive to the assumptions made regarding the form of the system considered for the
12 analysis. It can be seen that the failure probability of the connection (system) consisting
13 of one angle only is considerably higher than the case of assuming a two- or a four-
14 angle system. By comparing the system reliability profiles with those obtained earlier
15 through the traditional nominal stress (S-N) approach, it can be seen that a one-angle
16 system assumption gives more conservative remaining life estimates than its nominal
17 stress counterparts. The TCD method appears to result in a more rapid increase in the
18 probability of failure of the connection with time as compared to the nominal stress
19 method.

20 Having identified the potential crack initiation hot-spots in the riveted bridge connection
21 through the global-local analysis, the next step could involve a fatigue crack growth
22 analysis using Fracture Mechanics (FM) principles (see Fig. 1). This step could provide

1 more refined remaining fatigue life estimates for the connection and would assist in
2
3 effective planning of inspections and possible repairs. The crack growth analysis can
4
5 also be extended to a probabilistic framework in order to take into account uncertainties
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7 in initial defect sizes, crack growth and material parameters, inspection limits, etc.
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10 11 12 13 **5. Conclusions** 14

15 In this paper, a generic fatigue assessment methodology for riveted railway bridges was
16 presented that summarises the work carried out by the authors and their co-workers over
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18 a number of years. The methodology concentrates on the fatigue behaviour of the
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20 riveted stringer-to-cross-girder connections and is based on the finite element analysis
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22 of a typical short-span riveted railway bridge, representative of a large number of such
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24 bridges in the UK. The first step of the methodology is the global fatigue analysis of the
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26 bridge from which the most fatigue-critical bridge connections are identified on an S-N
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28 basis and remaining fatigue life estimates are obtained. The global analysis can be
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30 extended within a probabilistic framework by taking into account loading, material and
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32 model uncertainties in order to estimate reliability profiles for the most fatigue-critical
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34 bridge connection. The second step of the methodology consists of the stress analysis of
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36 a detailed finite element model of the previously mentioned fatigue-critical connection.
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38 The Theory of Critical Distances, which, rather than relying on nominal stresses,
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40 considers the entire stress field ahead of the notch, is employed for fatigue damage
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42 calculations. In addition to these calculations, this step can yield valuable information
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44 regarding the locations where fatigue cracking is most likely to occur and can then be
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46 followed by fatigue crack growth analysis.
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1 Parallel to the description of the methodology, typical results obtained from the analysis
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3 of a typical riveted railway bridge were also presented. It was found that fatigue damage
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5 under historical train loading was rather small. By contrast, damage in the past thirty or
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7 so years is accumulating at a much faster rate due the introduction of heavier freight
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9 trains. Fatigue detail classification, modelling uncertainty and load evolution was found
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11 to play a significant role for estimating the remaining fatigue life of old bridges.
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15 Comparison of the results obtained using the TCD with their more traditional, detail-
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17 specific S-N counterparts revealed that the latter can underestimate fatigue damage in
18
19 some cases by a factor of 3.5. The probabilistic analyses, using both the S-N and TCD
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21 approaches, highlighted a very large scatter in fatigue life predictions underlining the
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23 importance of adopting increased inspection and management plans for this class of
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25 bridges.
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33 Given the nature of the fatigue phenomenon and the unpredictability associated with
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35 long-term usage of these structures, these numerical results presented in this paper need
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37 to be validated through field observations. Although the remaining fatigue life estimates
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39 for individual connections appear reasonable, it has to be remembered that many
40
41 thousand of these connections exist in the network and a timely management of repair
42
43 and replacement needs to be in place well before the bridges reach the end of their
44
45 service life.
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53

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Figure captions

Figure 1 Flowchart of fatigue assessment methodology for riveted railway bridges.

Figure 2(a) Typical superstructure of a short-span riveted railway bridge.

Figure 2(b) Global FE model of a typical riveted railway bridge.

Figure 3 Results of full-scale fatigue tests on old steel and wrought iron riveted members and connections compared with three BS 5400 [1] S-N curves and the curve proposed by Xie et al. [23].

Figure 4 Fatigue test results on wrought iron riveted members and connections compared with the Railtrack [2] wrought iron mean and design S-N curves.

Figure 5 S-N based ranking of bridge connections (no DAF) [24].

Figure 6 Cumulative fatigue damage of bridge connection S7-S5.

Figure 7 Probability of fatigue failure versus time of bridge connection S7-S5 for two detail classifications and under a load evolution scenario [29].

Figure 8 Close-up view of the global-local FE model at a stringer-to-cross-girder connection location (connection S7-S5, see Fig. 2b).

Figure 9 Comparison of total fatigue damage between global and global-local models for two rivet clamping force values (period 1970-2007) [36].

Figure 10 Probability of fatigue failure versus time for different connection hot-spots and for different system assumptions [39].

Figure 11 Probability of fatigue failure versus time for different system assumptions [39].

Table 1

Period	Traffic Type	Locomotive Type	Wagon Type	Axle Weight	No of Wagons	Annual Frequency
1900-1920	F	0-6-0 Superheater Freight Engine		2×8 t	30	10500
	P	4-4-0 Passenger Engine		4×8 t	8	11250
	LS	0-4-4 Tank Engine		4×8 t	4	11250
1920-1940	F	0-6-0 Superheater Freight Engine		2×10 t	40	10500
	P	4-6-0 Superheated Mixed Traffic Engine		4×9 t	12	18000
	LS	0-4-4 Tank Engine		4×8 t	4	4500
1940-1970	F	2-8-0 Freight Engine		2×10 t	40	10500
	P	4-6-0 Superheated Mixed Traffic Engine		4×9 t	15	18000
	LS	0-4-4 Tank Engine		4×8 t	6	4500
1970-2007	F	Steel Train (no 1)		6×18.5 t	15	2257
	P	Diesel Hauled Passenger Train (no 5)		4×10 t	12	22500
	F	Heavy Freight Train (no 7)		4×25 t	10	2411
	F	Heavy Freight Train (no 8)		2×25 t	20	6027

Table 1 Rail traffic model (F=freight, P=passenger, LS=local suburban) [12].

Figure 1

Figure 1

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Figure 2

Figure 2(a)

Figure 2(b)

Figure 2(a) (tiff format)
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Figure 2(b) (tiff format)
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Figure 3

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Figure 9

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