Remaining fatigue life estimates for riveted railway bridges

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ABSTRACT
A large number of metallic railway bridges currently in use in the UK are of riveted construction and are close to 100 years old. A fatigue assessment methodology is needed for such bridges since these may be close to the end of their fatigue lives. Part of such a methodology is the global analysis of the bridge in order to identify the most fatigue critical details. The aim of this paper is to present results, in terms of Miner's damage at the riveted connections of a typical riveted UK railway bridge, obtained from a global finite element analysis under various train loadings.

KEYWORDS Fatigue Damage, Wrought-Iron Riveted Connections, FE Analysis

INTRODUCTION
A large number of riveted bridges can be found on the railway network in the UK and in other parts of Europe and North America. Continuously increasing loads and the fact that these bridges were not explicitly designed against fatigue raise questions regarding their remaining fatigue life. As is well known, metal fatigue exhibits relatively high levels of uncertainty and can be influenced by a number of structure- and environment-specific factors. Moreover, the load history, which plays an important role in fatigue life evaluation, is largely unknown in most cases. In view of the above, there appears to be a need to develop a comprehensive fatigue assessment methodology for riveted railway bridges.

There has been a considerable amount of research in order to study the fatigue behaviour of riveted bridges. This research has shown that fatigue damage in such bridges is mainly associated with secondary, rather than primary, stress effects [1-3]. As a result, riveted connections rather than the actual members themselves have been identified as the fatigue critical locations [1,2,4]. Indeed most of the cracks which have so far been found in riveted bridges are concentrated in the area of the riveted connections.

A typical riveted connection used in the construction of railway bridges is shown in figure 1. It consists of two angles riveted to the webs of the two connected members. Research has shown that this type of connection possesses considerable rotational fixity, enabling it to develop significant bending moment [1, 5]. The restrained out-of-plane deformation of the connection angle due to the rotational fixity, combined with the stress concentrations at the angle fillet, rivet holes, cope-holes being present and the rivets themselves have been identified as possible factors that lead to the fatigue cracking of riveted connections [1-3, 6].

The majority of the fatigue assessment methodologies for existing riveted bridges that have been developed in the past have been based on the S-N approach [7-10]. More elaborate methodologies based on the study of existing or assumed cracks using Fracture Mechanics have also been applied [11, 12]. Irrespective of the adopted methodology, selection of the fatigue critical detail(s) in a bridge needs to be addressed.

Here, in order to identify the fatigue-critical connections in a typical riveted UK railway bridge, a global finite element (FE) analysis of the bridge is carried out. The assumed loading consists of both nominal present-day and historical trains and volumes. The riveted connections are ranked according to their fatigue criticality which is calculated on an S-N basis. The effect of different parameters such as connection fixity, the choice of fatigue detail classification and dynamic amplification is investigated.

**FINITE ELEMENT ANALYSIS**

The riveted railway bridge under investigation is shown in figure 2. The superstructure of such bridges usually consists of longitudinal rows of railbearers (or stringers) interconnected with transverse cross-girders which in turn are connected to longitudinal main girders. Two types of connections can be identified in the figure; stringer-to-cross-girder connections denoted by S and cross-girder-to-main-girder connections denoted by C. All the members are built-up using plates, angles and rivets. This wrought-iron bridge has a 9.6 m span, was constructed at the beginning of the 20th century and is deemed to be representative of a large number of bridges on the UK network. The bridge is assumed to be simply supported on the three main girders.

![Diagram of riveted connection](image1.png)

**Fig. 1. Typical riveted connection [4]**.  **Fig. 2. Finite element model of the bridge.**

The bridge is modelled using the commercial FE-package ABAQUS 6.3 [13] using 8-noded shell elements for all members. A Young's modulus of 200 GPa, which is a typical value of UK-manufactured wrought-iron [14] and a Poisson’s ratio of 0.3 are used for the analyses. Particular emphasis is given to the modelling of the riveted connections between the members which are modelled using axial spring elements with variable stiffness.

The bridge live loading is assumed to consist of two railway traffic models representing two distinct periods. The loading from 1970 onwards is represented by the BS 5400 medium traffic model [15] whereas the loading in the period between 1900 and 1970 is represented by
a historical load model (table 1) which was developed by the authors in collaboration with Network Rail. As can be seen, this period is further divided into 3 parts that may be roughly associated with historical changes in rail traffic. The BS 5400 traffic model consists of a steel train (21.5t max. axle load), two freight trains (25t max. axle load) and a typical passenger train (20t max. axle load) [15].

Table 1. Historical load model (F=freight, P=passenger, LS=local suburban).

<table>
<thead>
<tr>
<th>Period</th>
<th>Traffic Type</th>
<th>Locomotive Type</th>
<th>Wagon Type</th>
<th>Axle Weight</th>
<th># of Wagons</th>
<th>Annual Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>1900-1920</td>
<td>F</td>
<td>0-6-0 Superheater Freight Engine</td>
<td>2×8 t</td>
<td>30</td>
<td>10,500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>P</td>
<td>4-4-0 Passenger Engine</td>
<td>4×8 t</td>
<td>8</td>
<td>11,250</td>
<td></td>
</tr>
<tr>
<td></td>
<td>LS</td>
<td>0-4-4 Tank Engine</td>
<td>4×8 t</td>
<td>4</td>
<td>11,250</td>
<td></td>
</tr>
<tr>
<td>1920-1940</td>
<td>F</td>
<td>0-6-0 Superheater Freight Engine</td>
<td>2×10 t</td>
<td>40</td>
<td>10,500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>P</td>
<td>4-6-0 Superheated Mixed Traffic Engine</td>
<td>4×9 t</td>
<td>12</td>
<td>18,000</td>
<td></td>
</tr>
<tr>
<td></td>
<td>LS</td>
<td>0-4-4 Tank Engine</td>
<td>4×8 t</td>
<td>4</td>
<td>4,500</td>
<td></td>
</tr>
<tr>
<td>1940-1970</td>
<td>F</td>
<td>2-8-0 Freight Engine</td>
<td>2×10 t</td>
<td>40</td>
<td>10,500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>P</td>
<td>4-6-0 Superheated Mixed Traffic Engine</td>
<td>4×9 t</td>
<td>15</td>
<td>18,000</td>
<td></td>
</tr>
<tr>
<td></td>
<td>LS</td>
<td>0-4-4 Tank Engine</td>
<td>4×8 t</td>
<td>6</td>
<td>4,500</td>
<td></td>
</tr>
</tbody>
</table>

For the purposes of the analysis, the trains are traversed in static steps of 1m over one track of the bridge. The axle loads are applied directly on the top flanges of the stringers thus neglecting any load spread due to the rails and sleepers. This assumption has been found to result in higher stresses in similar bridges [16]. Dynamic effects are accounted for by using amplification factors from structural codes and field measurements in similar bridges.

The resulting irregular stress histories, obtained at the riveted connections at distances of 250mm and 300mm from the stringer-to-cross-girder and cross-girder-to-main-girder interfaces respectively, are first converted into stress range blocks by using the rainflow counting method. The damage caused by a single train passage is calculated by applying Miner’s rule to the resulting stress ranges. Three different detail classifications are used for the riveted connections i.e. Class B, Class D and Class Wrought-Iron (WI). According to BS 5400, Class B is used for holes and can be assumed to represent the case of having low or no clamping force in the rivets. A stress concentration factor of 2.4 is used with this class [15]. Class D represents riveted lapped or spliced connections with normal or high clamping force. Lastly, Class WI is used to represent riveted wrought-iron connections as proposed by the UK railway assessment code [17] and can be considered as a more realistic case. The BS 5400 two-slope design S-N curves [15] are used for the first two classes when calculating the damage. The WI S-N curve lies between the modified Class B and Class D S-N curves. Consequently, Classes B and D can be regarded as upper and lower bounds for remaining
fatigue life estimates. Thickness effects, as proposed by BS 7608 [18], are neglected since the thicknesses of the members are in general slightly greater than 16 mm.

RESULTS AND DISCUSSION
In calculating the total damage of the riveted connections, the bridge is assumed to have been constructed in 1900. The connections are numbered as shown in figure 2. The notation, where the first symbol refers to the connection in question and the second symbol refers to the relevant direction, is used in the following sections. Previous studies have shown that an increase in the stiffness of the stringer-to-cross-girder connections results in an increase in the fatigue damage [19]. Consequently, all results presented hereafter are obtained by assuming fully fixed connections since this assumption provides conservative fatigue life estimates.

The frequency diagram shown in figure 3 depicts the total number of cycles versus the stress ranges experienced by connection S7-S5 under the historical load model and BS 5400 medium traffic assuming fixed connections. It can be seen that the maximum stress range experienced by the connection is approximately 45 MPa. All of the stress ranges are below the fatigue limit for an assumed Class D, while a very small number is above the fatigue limit for assumed Classes B and WI.

![Stress range frequency diagram for connection S7-S5 (period 1900-2004).](image)

Fig. 3. Stress range frequency diagram for connection S7-S5 (period 1900-2004).

**Effect of detail classification on fatigue damage**
The damage of the cross-girder-to-main-girder connections was found to be negligible irrespective of classification. On the other hand, the damage in the stringer-to-cross-girder connections, which is the most significant, is found to be axle-dominated with the trains having the highest axle loads (Trains No 7 and 8 of BS 5400) producing the highest amount of damage. The most highly damaged connections are found to be the inner stringer-to-cross-girder connections (S7-S5, S8-S6, S3-S5, S4-S6, see figure 2). Figure 4 depicts the cumulative damage as well as the remaining fatigue life of the two most highly-damaged connections (S7-S5 and S8-S6) for all three detail classifications. It can be seen that a Class B...
(no clamping force) leads to the highest damage with remaining fatigue lives of 80 and 85 years for the two connections. A Class D (high clamping force) results in remaining lives of 289 and 303 years. This comparison makes the effect of clamping force on the fatigue damage of riveted connections evident. The Class WI results can be seen to lie between their Class B and Class D counterparts. Damage ranking of the connection is found not to be significantly affected by the choice of the detail classification.

Figure 4 also depicts a considerable increase in the damage accumulation rate of the connections after 1970. This fact can be attributed to the introduction of the heavier axle-load BS 5400 traffic. The rate of damage increase is fairly small over the period 1900-1970 for all three classes. This increase is larger for Class B after 1970 and can be attributed to the stress range frequency box (see figure 3) which lies just above the Class B fatigue limit. This frequency box is a result of the BS 5400 heavy axle trains.

It can be seen in figure 4 that about 25 to 35% of the fatigue strength of connections S7-S5 and S8-S6 has been expended over the last 34 years for assumed Classes B and WI. These accumulation rates would rise even higher with the introduction of heavier axle trains in the future.

![Figure 4. Cumulative damage of connections S7-S5 and S8-S6 versus time.](image)

**Effect of dynamic amplification on fatigue damage**

Dynamic amplification, especially in short span, unballasted metallic railway bridges, is expected to have a considerable effect on the stresses experienced by the members and connections. Due to the sensitivity of fatigue damage on the stress ranges the effect of dynamic amplification on the damage of the connections is studied here.

Different dynamic amplification factors (DAF), obtained from structural codes and field measurements in similar bridges, are shown in table 2. These factors are applied to the statically calculated stress ranges. The factors obtained from Eurocode 1 (EC1) [20] and Network Rail [17] are very similar. The D23 factor is based on field measurement as well as
theoretical studies carried out by the European Rail Research Institute [21]. The remaining dynamic amplification factors were obtained from field measurements on short and medium span steel railway bridges [10, 22]. It can be seen in table 2 that there is a good agreement between the factors obtained from different sources. However, this is partly because the formulae providing these factors in the European codes are based on more or less the same studies.

Table 2. Dynamic amplification factors for short-span riveted railway bridge.

<table>
<thead>
<tr>
<th>Train speed</th>
<th>BS 5400 trains</th>
<th>Historical load model trains</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>72 km/h</td>
<td>80 km/h</td>
</tr>
<tr>
<td>EC 1 [20]</td>
<td>1.16</td>
<td>1.16</td>
</tr>
<tr>
<td>D23 [21]</td>
<td>1.16</td>
<td>1.18</td>
</tr>
<tr>
<td>Byers [22]</td>
<td>1.15</td>
<td>1.19</td>
</tr>
<tr>
<td>Tobias &amp; Foutch [10]</td>
<td>1.15</td>
<td>1.15</td>
</tr>
<tr>
<td>Network Rail [17]</td>
<td>1.14</td>
<td>1.16</td>
</tr>
</tbody>
</table>

The cumulative damage as well as the remaining fatigue lives for the most highly-damaged connection S7-S5 accounting for the dynamic amplification factors of table 2 and assuming Class WI is shown in figure 5. The case without consideration of any dynamic amplification (no DAF) is also illustrated on the figure. It can be seen that there is a considerable decrease in the remaining fatigue life of the connection due to dynamic amplification. The factor obtained from the D23 report [21] leads to the lowest remaining fatigue life with a decrease from 120 years, which is the case without any dynamic amplification, to only 21 years. The remaining factors result in similar remaining lives ranging between 26 and 31 years.

![Fig. 5. Cumulative damage of connection S7-S5 for different DAFs.](image-url)
Although the assumption that all stress ranges are multiplied by the same dynamic amplification factor is rather simplistic, it demonstrates that the remaining fatigue life estimates are very sensitive to the assumed dynamic amplification. This is due to the shift of the entire stress range frequency diagram to the right. This in turn results in the increase of some stress ranges that were previously below the fatigue limit to levels above the fatigue limit.

Other factors influencing fatigue damage
The effect of two trains passing over the bridge simultaneously has also been examined. It was found that the simultaneous passage of two trains can lead to an increase in the damage of the stringer-to-cross-girder connections by as much as 400% depending on whether the trains will enter the bridge at the same time or not. However, since the annual occurrence of this phenomenon is rather low, its effect on the total damage of the connections can be regarded as negligible.

Due to the significant variability in the material properties of older steels and wrought-iron [14, 23-25], the effect of Young's modulus on fatigue damage has also been briefly investigated. It was found that a decrease in the value of Young's modulus from 200 GPa to 170 GPa, the latter being a low but nevertheless encountered value [25], increases the damage of the stringer-to-cross-girder connections by as much as 510% depending on detail classification.

CONCLUSIONS
In this paper, a global finite element analysis of a typical riveted UK railway bridge was carried out in order to identify the most fatigue-critical connections. Total damage in the connections was calculated under the historical load model and the BS 5400 medium traffic and remaining fatigue life predictions were made. The effect of various parameters such as fatigue detail classification and dynamic amplification were investigated.

The global stress analysis has identified the fully-fixed, inner stringer-to-cross-girder connections as the most fatigue critical (S7-S5, S8-S6, S3-S5, S4-S6, see figure 2). The damage under the historical load model (1900-1970) was found to be small with the damage accumulation rate increasing due to the introduction of the heavier BS 5400 trains (post-1970). Dynamic amplification was found to result in considerable reduction in the remaining fatigue lives. A more detailed investigation of the most critical connections using Fracture Mechanics is currently underway.

ACKNOWLEDGEMENTS
The work described in this paper forms part of an ongoing project supported by EPSRC and Network Rail. The opinions expressed are those of the authors and do not necessarily represent those of the sponsoring organisations. We would like to thank Mr Brian Bell, Network Rail Project Officer, for valuable discussions and feedback on the methodology and results reported in this paper.

REFERENCES


